RAPID LOSS MODELING OF DEATH AND DOWNTIME CAUSED BY EARTHQUAKE INDUCED DAMAGE TO STRUCTURES

A Thesis

by

SANDEEP GHORAWAT

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

May 2011

Major Subject: Civil Engineering

RAPID LOSS MODELING OF DEATH AND DOWNTIME CAUSED BY EARTHQUAKE INDUCED DAMAGE TO STRUCTURES

A Thesis

by

SANDEEP GHORAWAT

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Approved by:

Co-Chairs of Committee,	John B. Mander
	Ivan D. Damnjanovic
Committee Member,	Scott Lee
Head of Department,	John Niedzwecki

May 2011

Major Subject: Civil Engineering

ABSTRACT

Rapid Loss Modeling of Death and Downtime Caused By Earthquake Induced Damage to Structures.

(May 2011)

Sandeep Ghorawat, B.Tech., Indian Institute of Technology Guwahati, India

Co-Chairs of Advisory Committee: Dr. John B. Mander
Dr. Ivan D. Damnjanovic

It is important to assess and communicate the risk to life and downtime associated with earthquake induced damage to structures. Thus, a previously developed four-diagram/four-step approach to assess direct losses associated with structural damage, a similar quantitative risk assessment technique is used to examine the indirect loss associated with death and downtime. The four-step approach is subdivided into four distinct tasks: (a) Hazard analysis, (b) Structural analysis, (c) Loss analysis of both direct and indirect losses and (d) The total loss estimation due to damage, death and downtime. This empirically calibrated model in the form of power curve is used by establishing losses corresponding to onset of damage state 5 (complete damage) and limiting upper losses. The utility of the approach is investigated for the bridges in both California and New Zealand regions with different detailing. Results show that death related losses for bridges are generally twice and downtime five times the direct damage losses. Thus, it is concluded that structures should be designed for more than just acceptable physical damage. It is shown that a marked improvement can be made by moving to a comprehensive damage avoidance design paradigm.

DEDICATION

To my Family and Friends

ACKNOWLEDGEMENTS

I thank my co-chair, Dr. Mander, for his invaluable guidance and support throughout the course of this research. This work would not have been possible without his vision and thoughts.

I would also like to thank my other co-chair, Dr. Damnjanovic, and my committee member, Dr. Lee, for serving on my advisory committee. Thanks also to the department faculty and staff for making my time at Texas A&M University a great and memorable experience. I would also like to thank Angad, Bharath, Pankaj, Sireesh, Vivek and Yamshi at Texas A&M University and elsewhere for their constant support and encouragement.

Last but not the least, I would like to show my gratitude towards my family for making this unforgettable journey at Texas A&M University possible and for their constant support, prayers and encouragement.

TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

1. INTRODUCTION

1.1 Background and Motivation

Earthquakes are one of the most hazardous natural events which may cause devastation without warning. Losses due to these types of catastrophes can be characterized in terms of the 3D's: *Damage, Death and Downtime*. Performance-based earthquake engineering (PBEE) consider these 3D's as *Performance* measures. As such they should also be addressed in loss estimation procedures as the repair cost will not only be the direct "loss" suffered damage to the owner, but also indirect losses to users in terms of death and downtime. Thus total losses strictly represent the sum of both direct and indirect losses that necessitate repair or rebuilding due to earthquake effects. Generally, only implied losses from structural damage are considered for the design of infrastructure.

The conventional definition of risk is the product of the probability of the hazardous event and its associated consequences (Stewart 2009). This definition is consistent with that used by the US Department of Homeland Security (DHS) National Infrastructure Protection Plan where risk is assessed from any scenario as a function of consequence, vulnerability, and threat (DHS 2009). Thus it is important for stakeholders to know potential downtime and potential fatalities due to collapse of the facility along with damage repair/replacement costs. This will help a responsible owner to mitigate the risk to the greatest extent possible.

This thesis follows the style of *Journal of Structural Engineering*.

Recently, Mander and Sircar (2009) and Sircar et al. (2009) developed a four-step approach to assess the direct financial losses arising from structural damage to constructed facilities. Although the approach is general and could be applied to any type of hazard, Sircar et al. (2009) focused on earthquake hazards. Moreover, it is considered that the four-step approach can be extended to encompass death and downtime. This will enable the calculation of total "3D-losses" as the model follows similar steps. The objective of using the Mander and Sircar (2009) direct four-step approach for computing losses is to relate estimated losses in terms of well-known seismic demand and structural capacity factors. The loss estimation framework is divided into four interrelated steps: (a) hazard analysis, (b) structural analysis, (c) damage/loss analysis; and (d) loss estimation. When these losses are integrated over all possible seismic scenarios, the Expected Annual Loss (EAL) can be computed directly in terms of a simple formula.

In this research the direct four-step analysis process is extended for the estimation of death and downtime losses. The approach to incorporate the indirect losses of death and downtime mirrors what Mander and Sircar (2009) did for physical damage loss estimation as follows: First, an intensity measure (IM) of hazard such as acceleration is related to its rate of occurrence (or annual frequency f_a or return period for earthquakes). Second, the earthquake IM is related to the response of the structure via engineering demand parameters (EDP) such as structural drift (θ); this depends on the type of structure and its design detailing. These first two steps are common in estimating each of the 3D's. The third step is to associate response of the structure with corresponding losses. This can be performed by integrating vulnerability curves over various damage states to corresponding losses in terms of each of the 3D's: Damage, Death and Downtime. The fourth and final step is to associate losses with the hazard frequency; this can be easily performed by relating the first three steps using a single compound formula as proposed by Mander and Sircar (2009). The expected annual losses for the 3D's can be estimated by integrating losses from their onset over all possible scenarios.

The objective of this thesis is to develop loss models for Death and Downtime as an extension of the Mander and Sircar (2009) approach for Damage. Examples will be drawn from bridges of different design standards and seismic regions.

1.2 Literature Review

1.2.1 Seismic Hazard, Risk and Loss

Low probability-high consequence events like earthquakes have a potential to cause losses, both structural and non-structural in terms of life, property damage and facility downtime. It is equally important to predict and mitigate the losses over the period of time. A predictive capability should help in preparing for the worst to come and inform owner/users about risk to the facility. As physical damage is a direct loss, it has always been a primary focus for engineers and others to study the consequences of a hazardous event. Determining the risk at the site of structural project is the first step in a performance based design.

Cornell (1968) introduced a method for the evaluation of the seismic risk in terms of a ground motion parameter (peak ground acceleration) versus average return period at the site of an engineering project. This was a first step in the direction of a probabilistic risk assessment methodology. Later, Algermissen (1972) studied losses in the San Francisco Bay area so as to provide the California Office of Emergency Services on the possible losses produced by large, damaging earthquakes. This was when California Office of Emergency Services started thinking of some rational basis for state rescue and recovery operations for future.

Kennedy et al. (1980) studied probabilistic seismic safety of Oyster Creek nuclear power plant. They considered earthquake hazard as an initiating event that could result in radioactive release based on probability of earthquake and probability of failure because of the event. Then, Kennedy and Ravindra (1984) extended their work on nuclear power plant risk studies using seismic fragilities. They developed seismic fragilities of critical structures and equipment as families of conditional failure frequency curves plotted against peak ground acceleration to use in a probability risk assessment. But sensitivity studies need to be conducted to judge the influence of different assumptions on risk estimates. Sometime later, Kennedy (1999) worked on risk based seismic design criteria by aiming certain desired seismic risk in terms of an annual probability of seismic-induced failure. The integral part of the framework was to establish the acceptable seismic margin above the design Safe-Shutdown-Earthquake (SSE) response spectrum. Garrick and Christie (2002) practiced probability risk assessment technique on nuclear power plants in USA. This signaled the beginning of a revolution in the licensing process of commercial nuclear power plants.

In 1989, the U.S. Federal Emergency Management Agency (FEMA) prepared a report on estimating losses from future earthquakes and presented it to the National Academy of Science (NAS). This report may be considered as a cornerstone for carrying out loss estimation method development and studies. The National Institute of Building Sciences (NIBS 1994) assessed the state-of-the-art earthquake of loss estimation methodologies. Thereafter, FEMA collaborated with National Institute of Building Science (NIBS) and started developing a standard nationally applicable seismic loss estimation framework on a regional basis (Whitman et al. 1997). The framework is developed and embodied in geographical information system (GIS) MapInfo-based software called HAZUS.

HAZUS (1997) was the first edition of risk assessment software package built on GIS technology, used for mapping and displaying hazard data and the results of damage and economic loss estimates for building and infrastructure. Kircher et al. (1997) used building damage functions developed by Whitman et al. (1997) for earthquake loss estimation. These functions estimate the probability of discrete states of structural and nonstructural building damage and hence estimate building losses. Thereafter, Mander and Basoz (1999) developed the seismic fragility curves for highway bridges through the use of rapid analysis procedures. This was based on fundamentals of mechanics and dynamics and data obtained from the National Bridge Inventory. The fragility curves were used to associate losses in terms of its discrete damage states.

HAZUS (2003) was developed as an upgraded version of HAZUS (1997) which can assess potential losses from multi hazards like floods, hurricanes and earthquakes.

Kircher (2003) described procedures based on and compatible with HAZUS that may be used to develop earthquake damage and related loss functions for welded steel moment-frame buildings. But an experienced structural engineer is needed to carry out the process.

Shome and Cornell (1999) developed a relationship between seismic demands on structures in terms of ground motion parameters which is part of the second step of the performance based design model. They worked on probabilistic seismic demand analysis of nonlinear structures in terms of ground motion parameters and frequency of earthquake. Thereafter, Cornell and Krawinkler (2000) described the foundation on which performance assessment is based and the major challenges like defining the objectives of seismic performance assessment (to go for general methodology for estimating the annualized expected costs or the structure should be compatible for retrofit which leads to evaluation of design alternatives) on the way to expected success.

Porter et al. (2001) worked on assembly-based vulnerability framework for evaluating the seismic vulnerability and performance of building. The framework applies the ground motion time history to structural model to determine structural response and then utilizes the damage to individual building components to estimate the total loss. The framework and simulation approach is fully probabilistic and addresses damage in a highly detailed manner and is building specific. The approach does not rely extensively on expert opinion but this comes with a cost of rigorous structural analysis and creating building assemblies and their capacity. Later on, Vamvatsikos and Cornell (2002) proposed the Incremental Dynamic Analysis (IDA) method, which offers thorough seismic demand and capacity prediction capability by using a series of nonlinear dynamic analyses under a multiply scaled suite of ground motion records. But selecting a suite of earthquakes at the desired location/distance from fault is very important as results may largely depend on it.

Porter et al. (2004) worked on economic seismic risk estimation for buildings using three different ways; 1) Integration of seismic vulnerability and hazard, 2) Probable frequency loss and 3) Linear assembly-based vulnerability (LABV). LABV is similar to four-step rapid modeling process with a striking difference as it simplifies the analysis of the 50-year loss using linear spectral analysis. They expressed that probable maximum loss (PML) has no place in a standard financial analysis and need to replace with more meaningful and applicable term like probable frequent loss (PFL) or expected annual loss (EAL). They estimated EAL using linear assembly-based vulnerability (LABV) for number of wood-frame buildings.

Goulet et al. (2007) evaluated seismic performance assessment in terms of economic losses and collapse safety of a reinforced concrete moment frame building designed for building code provision (2003). Their work suggests that while considering higher hazards levels, it is important to consider response spectral shape otherwise it leads to overestimation of mean annual collapse rate by a factor of 5-10. Later, Baker and Cornell (2008) worked on determining aleatory and epistemic uncertainty (inherent randomness and model uncertainty) in each stage of seismic loss estimation framework and its propagation through the analysis. Mackie et al. (2009) proposed improvement on previous bridge loss models by local linearization of the dependence between repair quantities and damage states so that the resulting model follows a linear relationship between damage states and repair points. But this becomes more complicated when the structure is large and complex.

Dhakal and Mander (2006) expanded upon the idea of the Pacific Earthquake Engineering Research (PEER) triple integral to include losses in a total probabilistic framework. Their resulting quadruple integral led to the quantification of the expected annual financial loss (EAL) for engineered facilities for any natural hazard. At the same time, Dhakal et al. (2006) worked extensively on seismic financial risk analysis of reinforced concrete buildings and demonstrated major shortcomings in existing construction practice. They also showed that improved detailing gives a significant economic payback in terms of drastically reduced financial risk.

Solberg (2007) performed experimental and computational tests on precast concrete structures designed for damage avoidance. DAD structures accommodate non-linear behavior by rocking at specially detailed connections and unbounded prestress is employed to provide restoring force and supplemental devices are used to dissipate energy. The EAL of a bridge pier designed for damage avoidance is approximately 25 percent of that of a conventional ductile. Subsequently, Mander et al. (2007) applied incremental dynamic analysis (IDA) to investigate expected structural response, damage outcomes, and financial loss from bridges. The results showed that ductility may have some effect on seismic safety of New Zealand bridges but it is not an alternative to strength. Later, Solberg et al. (2008) presented a simplified method for EAL without conducting non-linear dynamic analysis. They proposed a Rapid IDA-EAL method. This was shown to be a powerful, yet simple approach and shows good agreement with the more comprehensive but time consuming computational IDA-EAL method. They applied the model to two highway bridge piers; one traditionally designed for ductility and the other based on DAD.

Bradley et al. (2007) improved seismic hazard model for use in performance-based earthquake engineering and illustrated the significance of the proposed model on a typical bridge via probabilistic seismic demand analysis. They have also considered propagation of epistemic uncertainty in the seismic hazard and compared the model with power law relationship to illustrate its effects on the risk assessment. The drawback of the model was that it does not model the response in the region of global collapse.

Mander and Sircar (2009) and Sircar et al. (2009) developed a four-step approach to assess the direct financial losses arising from physical damage to constructed facilities. They observed that four-step probabilistic loss model is visually interrelated through log-log graphs. These losses for each damage states take into account epistemic uncertainty as well as the effect for loss (cost, downtime and death) surge following a major hazardous event. The model considered 30% price surge due to inflation in prices of material and labor in the wake of devastating earthquake. They also considered 10% swing in various parameters to estimate the change in estimated annual losses per million dollars of assets. Their inter-relationship via power equations leads to straight lines between specified coordinates. The four steps can be unified into one single compound equation which takes the generalized form. Full details of this model are given in Section 2 and Section 3.

1.2.2 DAD

One way to minimize physical damage losses is to construct the structure differently from the customary use of ductile details. Damage Avoidance Design (DAD), an emerging seismic design paradigm, was first proposed by Mander and Cheng (1997). In DAD, damage is avoided by special detailing of the column connections and it behaves in bilinear fashion with neither damage nor degradation in strength and stiffness. This is also discussed in Mander et al. (2007) and experimentally verified in Hamid and Mander (2006). They applied the new design philosophy to the design of bridge substructures. Ajrab et al. (2004) showed that DAD can be a really productive design to counter losses due to earthquakes as it is based on rocking base principle that provides extra damping to absorb and dissipate seismic energy effectively. Solberg et al. (2008) and Sircar et al. (2009) showed that DAD reduces the damage to bridges and consequently limit the physical losses due to earthquakes.

1.2.3 Fatal Accident Rate (FAR)

Hazards can lead to loss of life and it is important to know the frequency and consequences of the same time. Kletz (1971) introduced a term called Fatal Accident Frequency Rate (FAFR) as a measure of risk related to an activity, which is now widely known as Fatal Accident Rate (FAR). He carried out hazard analysis as the Health and Safety at Work Act requires UK employers to provide a safe plant and system of work as far as is reasonably practicable. FAR was developed as a basic measure of risk and it can be heuristically thought as fatalities per 1000 people over a period of 40 years of working life. Later on, nuclear and chemical industry started taking risk analysis and hazard assessment seriously (Lawley and Kletz (1975); Gibson (1976); Griffiths (1978); Dunster and Vinck (1979); Lawley (1980); Kennedy et al. (1980); Kennedy and Ravindra (1984)).

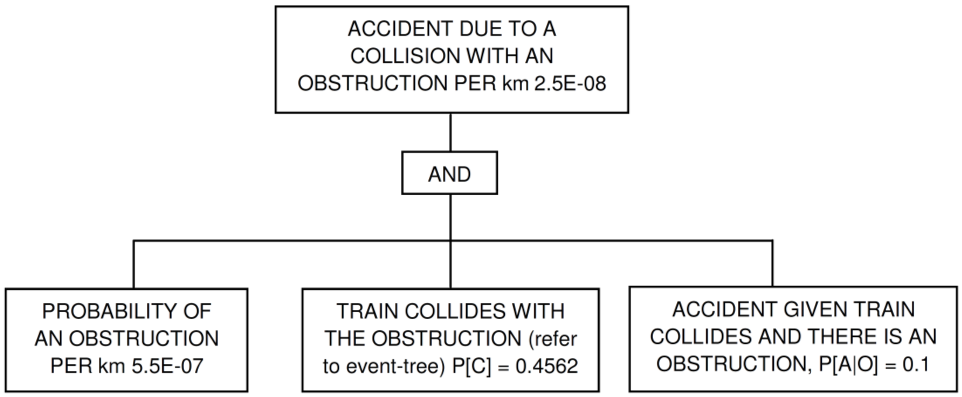
Kletz (1978), Lees (1980) and Elms and Mander (1990) developed some typical FAR values for various activity or risk exposure. They explained that risk exposure depends on the activity of a person at the time of disaster. Elms and Mander (1990) used FAR as a measure of risk to railway locomotive engineers who are exposed to recurring hazards while driving trains and are affected by various hazards such as slips, floods, speeding, signal, mechanical and track faults and sleep deprivation.

Subsequently, Mander and Elms (1994) worked on quantitative risk assessment of large structural systems. They used multiple fault and event trees to evaluate the probability of fatality from bridge and building collapse due to a catastrophic earthquake. They applied a similar analogy in various situations like locomotive collision with an obstruction (Figure 1.1). Even a sleeping person has a latent risk exposure and this has a measured value of FAR = 1.

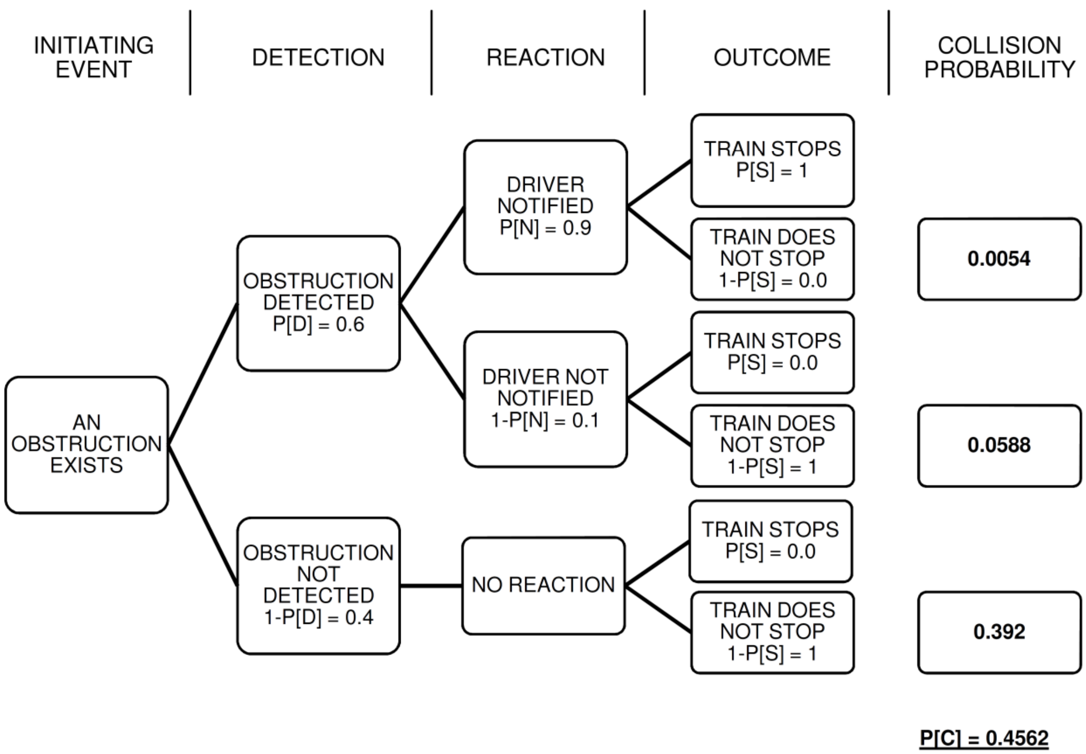
1.2.4 Value of Statistical Life (VSL)

Rice and Cooper (1967) estimated value of human life expressed in terms of lifetime earnings. It was calculated to compare the social benefits associated with investments in particular programs such as the highway construction, accident control, education, flood control etc. Later, Acton (1976) and Jones-Lee (1976) measured the monetary value of life saving programs as economic analysis. Thaler and Sherwin (1976) and Viscusi (1978) viewed value of saving a life using evidence from the labor market. Rhoads (1978) calculated how much should be spent to save a life through various models like: 1) Discounted future earnings and 2) Willingness to pay. Linnerooth (1979) reviewed the models to estimate the value of human life and suggested some drawbacks and modification for each model. Henley and Kumamoto (1992) mentioned the outcome of risk study that as risk diminishes to less than 10^{-6}/ year, the average individual does not show undue concern, and so elaborate precautions against this risk level are seldom taken. For example, this is roughly the probability that a person can be hit by lightning.

Kniesner and Leeth (1991) studied effect of institutionalized wage setting by comparing estimated wage differentials for fatal injury risk in Australia, Japan and the United States. Later, Miller (2000) studied the variations in values Value of Statistical Life (VSL) between different countries. Although it is true that life is priceless and putting a price tag may seem inhumane, it is necessary to do so to quantify the significance of human losses for comparative purposes to other direct and indirect losses like damage and downtime. In statistical terms, VSL is the cost of reducing the number of deaths by one. There are different types of approaches various groups use to evaluate VSL.



(a) Fault Tree



(b) Event Tree

Figure 1.1: Fault and event trees for a locomotive collision with an obstruction (Mander and Elms 1994).

The U.S. Department of Transportation (2007) has suggested VSL = $5.8 million. Such a VSL is a mean value of various studies carried out by five different people over the period of four years. Another governmental agency, Environmental Protection Agency (EPA) has suggested that VSL = $6.9 million in 2008. Kniesner et al. (2009) examined the differences in the VSL across potential wage levels in panel data using quantile regressions with intercept heterogeneity. Their findings indicate that a reasonable average cost per expected life saved cut-off for health and safety regulations in $7 million to $ 8 million. Based on the above, as of the time of writing a value of VSL = $ 6.0 million shall be used in this work.

1.2.5 Downtime

Basoz and Mander (1999) developed fragility curves for the highway transportation lifeline module of HAZUS. They developed a complete description on the theoretical background of the damage functions and associated each damage state with corresponding downtime. From this downtime losses could be assessed. Beck et al. (1999) developed a methodology modified from a repair-time model to estimate the rational component of downtime. The remaining portion of structure downtime is difficult to model because it is highly dependent on irrational components.

Comerio (2000) expressed that downtime includes the time necessary to plan, finance and complete repair of facilities damaged in earthquakes or other disasters. She goes on to point out there are various factors that can affect downtime: structural inspection, damage assessment, finance planning, architect/engineering consultations, a possible competitive bidding process, and the repair effort need to return a structure back to its undamaged state (Comerio 2006).

The rational components of facility downtime are attributed to the time needed to repair building damage. This is also an essential part of loss modeling, because it is one measure of operational failure in a lifeline or the business interruption loss in buildings. Comerio considered the University of California Berkeley as an experimental region and the losses after the 1994 Northridge earthquake were used to calibrate her models. A simplified method for estimating downtime was developed based on the floor area of buildings.

1.3 Scope of Thesis

The scope of the thesis is outlined below:

i. To develop the empirically calibrated four-step model for estimating annual losses in terms of expected death and downtime based on the four-step approach of Mander and Sircar (2009).

ii. To calculate FAR and estimate annual death losses in monetary terms using VSL. Also, to associate downtime with financial losses for different kinds of bridge structures.

iii. To prepare DAD bridge design according to California design code and seismicity and its response for a given intensity measure in terms of drift.

iv. For a prototype bridge, compare and aggregate the 3D losses from different designs and detailing (non-seismic, seismic and DAD) using examples based on California and New Zealand seismicity along with Japan design standards.

1.4 What Then Is Particularly New in This Thesis?

The particularly new work presented in what follows is outlined below:

i. Historically, engineers have fixated on quantifying the extent of physical damage losses through fragility analysis; death and downtime have only been paid scant attention. This thesis seeks to redress this imbalanced view.

ii. As it is important to develop a common and easy method to estimate death and downtime losses along with damage, the four-step approach proposed by Mander and Sircar (2009) has been extended accordingly.

iii. To compare 3D (death, downtime, damage) losses on one scale in order to study the relative importance of each. VSL and downtime cost in terms of annualized dollar losses is used to convert death and downtime into monetary terms, respectively.

iv. The extended 3D seismic loss analysis is applied to the bridges studied by Solberg et al. (2008). Thus examples of historic non-seismically designed structures along with conventionally designed ductile seismic resistant structures and the emerging damage avoidance design (DAD) class of structures are investigated.

2. RAPID LOSS MODELING OF FATALITIES CAUSED BY SEISMICALLY DAMAGED STRUCTURES

2.1 Summary

Structural design codes and specifications are primarily concerned with preserving life-safety. But these are not accordingly calibrated in a direct probabilistic risk or life-safety context. In this section a probabilistic fatality rate framework is developed for structures where seismic hazard is related to structural response, which is then related to damage and collapse, which in turn is related to the potential for fatality. The model is a power curve and calibrated using event and fault trees. The power curve is copped with lower and upper bounds, the former relates to damage onset while the latter corresponds to complete damage (damage state 5, toppling or collapse). The utility of the approach is investigated for bridges and the calibrated model is validated with Caltrans, Japan, New Zealand bridges along with Damage Avoidance Design (DAD), Seismic and Non-seismic designed structures. Result shows that Fatal Accident Rate (FAR) for DAD is very low while for non-seismically design structures it is seven times higher. The results are then converted into monetary terms using the value of statistical life (VSL).

2.2 Introduction

Failure of an engineered structural system may lead to direct and collateral damages, that is: physical damage to the constructed facility; loss of life or limb; and down-time within the system leading to loss of revenue and profit. Often engineers disregard the consequences of failure while focusing on the preservation of life-safety via collapse prevention for a maximum considered design event such as an earthquake with a return period of say 1000 years. It is important to not only communicate the risk to life and limb associated with structural damage but stakeholders also need to know the indirect financial losses along with the long-term economic losses arising from downtime. The objective of this section is to develop a simplified procedure that directly relates hazard intensity to response of the structure through collapse and hence to the chance of fatality.

A quantitative risk assessment technique is proposed to examine the risk to life and computed in terms of the well-known Fatal Accident Rate (FAR). A four-step approach is used which can be subdivided into four distinct tasks; (a) hazard analysis; (b) structural analysis; (c) damage and hence fatality analysis; and (d) FAR estimation. Recent research has shown that combination of fragility curves with loss functions can be used for probabilistic risk assessment methodology to estimate expected annual financial loss for structure (Kircher et al. 1997; Dhakal et al. 2006; Mander et al. 2007; Solberg et al. 2008). More recently a direct rapid loss assessment approach has been proposed by Mander and Sircar (2009) and Sircar et al. (2009) for earthquake induced damage to buildings and bridges.

The first step involves determination of seismic hazard at the constructed facility site by developing a relationship between earthquake intensity measures (IM) and its annual frequency, the inverse of which is return period. Such a model needs to be based on all seismic hazards (and thus faults) at a site, and therefore the general seismicity of a region with predictions based on existing historic catalogue information and models. Similarly, the second step relates IM with structural response in terms of engineering demand parameters (EDP) such as drift (θ). The third step involves associating EDP with probability of fatality using fragility curves. This is accomplished by associating damage states with an EDP in terms of drift and later on, associating losses at those drifts. The fourth and final step is inter-relating the first three-steps and integrating losses over the entire range of frequency. Then these losses can be converted into the well-known parameter, FAR. Each of these relationships involves uncertainty and must be treated probabilistically from location, seismic demand versus capacity, and capacity versus fatality.

FAR is the common measure to describe potential for fatalities and it can be thought as number of fatalities per 1000 lives over a period of 40 years due to an event or activity with 2500 hours every year. It is transient in nature as exposure changes from structure to structure and activity to activity. General risk for a specific activity is given by a composite FAR. This can be disaggregated into components of the risk which may arise from different failure modes or accident types. There is need to develop a method that relates probability of fatality to an EDP through a simple relationship in order to rapidly determine the probability of a fatality for a specific scenario earthquake event as well as overall FAR.

The aim of this work is to develop a closed form fatality estimation framework that directly relates (a) hazard to (b) structural response to (c) damage and losses (d) and hence to fatality probability. Note that customary evaluation of convolution integrals is not needed in this direct approach. The proposed closed form framework for the fatality estimation procedure is derived extensively from recent work by Mander and Sircar (2009) along with quantitative risk assessment work by Mander and Elms (1994) that produced fatality estimates.

The present section applies the framework to the failure of transportation structures, specifically bridges designed to different specifications and more specifically, contrasting non-seismic, seismic and emerging DAD philosophies.

2.3 Direct Loss Estimation Framework

This framework is used by Mander and Sircar (2009) for financial loss analysis of seismically damaged buildings. It is a quantitative four-step process to estimate the expected annual losses for different types of structure. Figure 2.1 shows the four-step framework along with its connectivity between each step. The inter-relationships may be approximated as capped linear functions in log-log space. The main objective of using the direct four-step procedure is to relate the probability of loss of life with well-known seismic demand and structural capacity parameters.

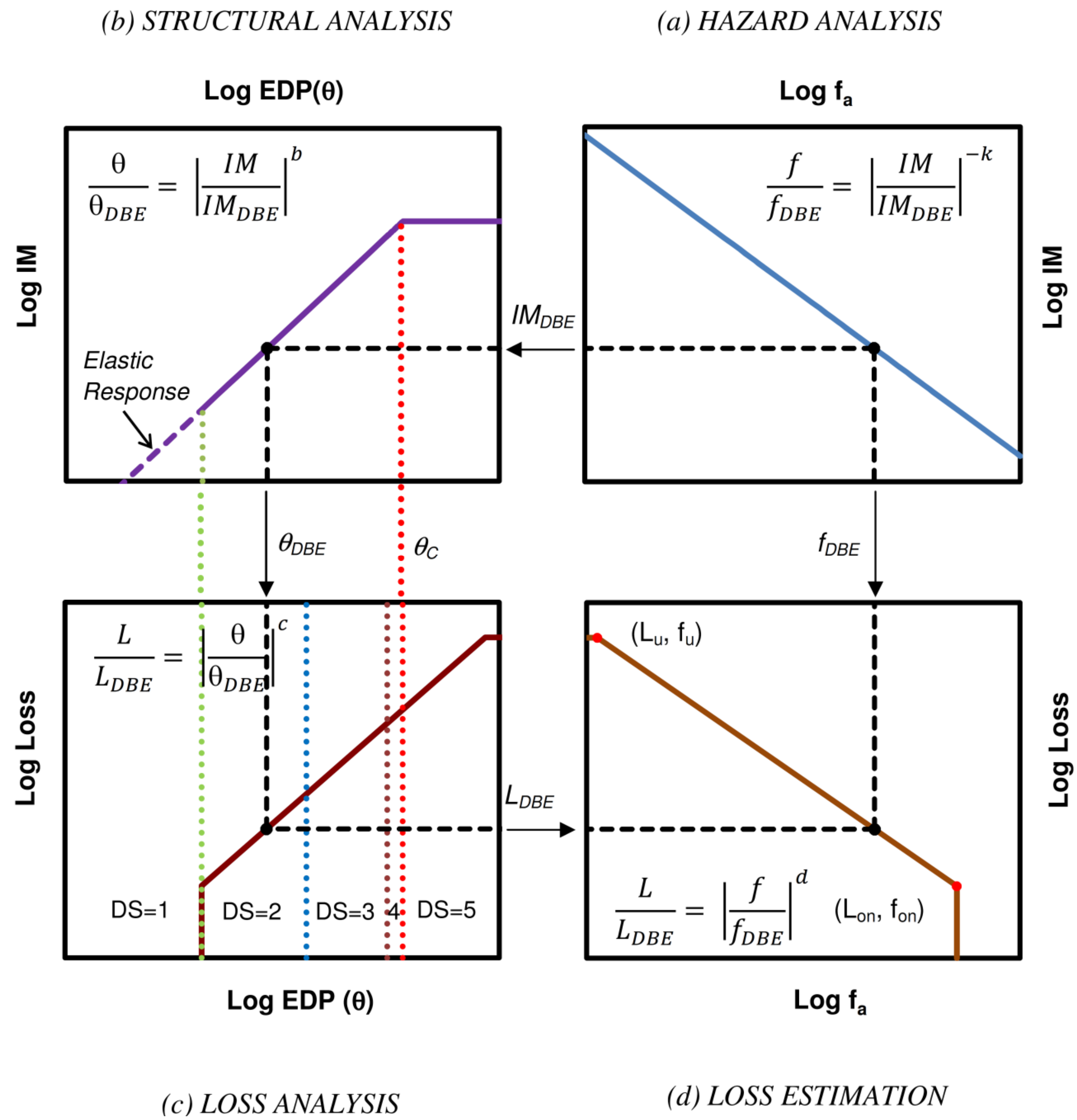


Figure 2.1: Summary of the Four-step approach used to estimate loss. (a) Two points on hazard recurrence curve are used to compute the IM (hazard analysis). (b) The IM's derived from (a) Are used to compute inter-story drifts using the hazard-drift curve (structural analysis). (c) The drifts obtained from (b) Are used to compute death loss (d) From which loss is computed.

In order to estimate expected annual loss ratio, it is essential to provide a well-defined relation between intensity measure (*IM*) and annual frequency (f_a), referred to herein as the seismic hazard-recurrence relationship. From Figure 2.1(a) a graphical representation of the relationship between hazard recurrence rate and the intensity measure is provided. As seen, the power relationship is plotted as straight line on log-log plot through two points and this represents a suitable approximation of hazard analysis. The relationship between them has been well-defined previously (Kennedy 1999; Cornell et al. 2002; Solberg et al. 2008).

Cornell et al. (2002) proposed a similar power relationship for structural analysis between IM and Engineering Demand Parameters (EDP). Figure 2.1(b) thus presents straight line relationship in log-log space for IM and EDP in the form of drift θ.

The relationship governing the four-step model and which gives mutual relation between four graphs is (Mander and Sircar 2009):

$$\frac{L}{L_{DBE}} = \left|\frac{\theta}{\theta_{DBE}}\right|^{c} = \left|\frac{S_a}{S_{a\,DBE}}\right|^{bc} = \left|\frac{f_a}{f_{DBE}}\right|^{\frac{bc}{-k}} \tag{2.1}$$

where f_a = annual frequency; S_a = spectral acceleration; f_{DBE} and $S_{a\,DBE}$ are the annual frequency and spectral acceleration demand (an IM) for design basis earthquake (DBE), typically taken as 10 % in 50 years or f_{DBE} = 1/475; θ = column (or interstory) drift on the structure for the considered event; θ_{DBE} = interstory drift on the structure for DBE; and b = exponent that represent slope of the line on log-log plot for Figure 2.1(b); c = constant, equal to the slope (in log-log space) of the line in Figure 2.1(c) and it is determined from empirical calibration of frequency with fatality curve; k = empirical seismic hazard constant; L = physical damage loss ratio; L_{DBE} = losses corresponding to design basis earthquake. The exponent of Figure 2.1(d), d, is inter-related to the first three powers as:

$$d = \frac{bc}{-k} \tag{2.2}$$

They proposed two-parameter power curve with upper and lower cutoffs to represent a loss ratio as a function EDP. The empirical model is expressed as:

$$\frac{L}{L_c} = \left|\frac{\theta}{\theta_c}\right|^c \; ; \; L_{on} \leq L \leq L_u = 1.3 \tag{2.3}$$

in which θ = column lateral drift (the EDP); $\theta_c = f\ \theta_{DB5}$ critical drift (where f = adjustment factor for low damage structures where $f > 1$, generally $f = 1$, but this may have a different value for certain special structure types; and θ_{DB5} = drift value for onset of complete damage); L = loss ratio for a given drift (θ); L_c = unit cost (normally $L_c = 1$, at onset of complete damage; Damage State 5); L_{on} = loss ratio at onset of damage state 2; L_u = loss ratio at complete damage or toppling of structure (30% more to allow price surge).

2.4 Proposed Death Loss Model

Previously Shiono et al. (1995) showed that fatality follows a log-log linear (power) relationship with collapse rate of building. Thus fatality model developed uses similar type of power relationship as it is been developed for physical damage losses (Mander and Sircar 2009) and shown in Figure 2.2 along with various dispersion factors. Briefly, the four-step death loss model directly estimates losses due to inter-relationships between (a) hazard, (b) structural response, (c) damage and (d) death loss:

$$\frac{DL}{DL_{DBE}} = \left|\frac{\theta}{\theta_{DBE}}\right|^{c} = \left|\frac{S_a}{S_{a\,DBE}}\right|^{bc} = \left|\frac{f_a}{f_{DBE}}\right|^{\frac{bc}{-k}} \tag{2.4}$$

in which DL = probability of death loss; DL_{DBE} = probability of death loss at design basis earthquake.

As the probability of death loss associated with each damage state is not distinctly defined, so the model is calibrated using probable death loss at the onset of Damage State 5 (complete collapse or toppling). The model is then bounded with upper and lower cutoffs based on structural drift. Figures 2.3 and 2.4 uses fault and event trees to analyse the probability of death loss due to a catastrophic earthquake for bridges and buildings, respectively. Chance of fatality at onset of complete damage (drift corresponding to onset of Damage State 5) is taken as 10% for building and bridges (Mander and Elms 1994). The empirical death loss model with upper and lower cutoff takes the form as shown:

$$\frac{DL}{DL_c} = \left|\frac{\theta}{\theta_c}\right|^{c} and;\ DL_{on} \le DL\ \le DL_u = 0.75 \tag{2.5}$$

in which DL_c = probability of death loss for drift corresponds to the onset of complete damage; DL_{on} = probability of death loss at onset of Damage State 2; DL_u = ultimate probability of death loss (at complete damage or toppling of structure).

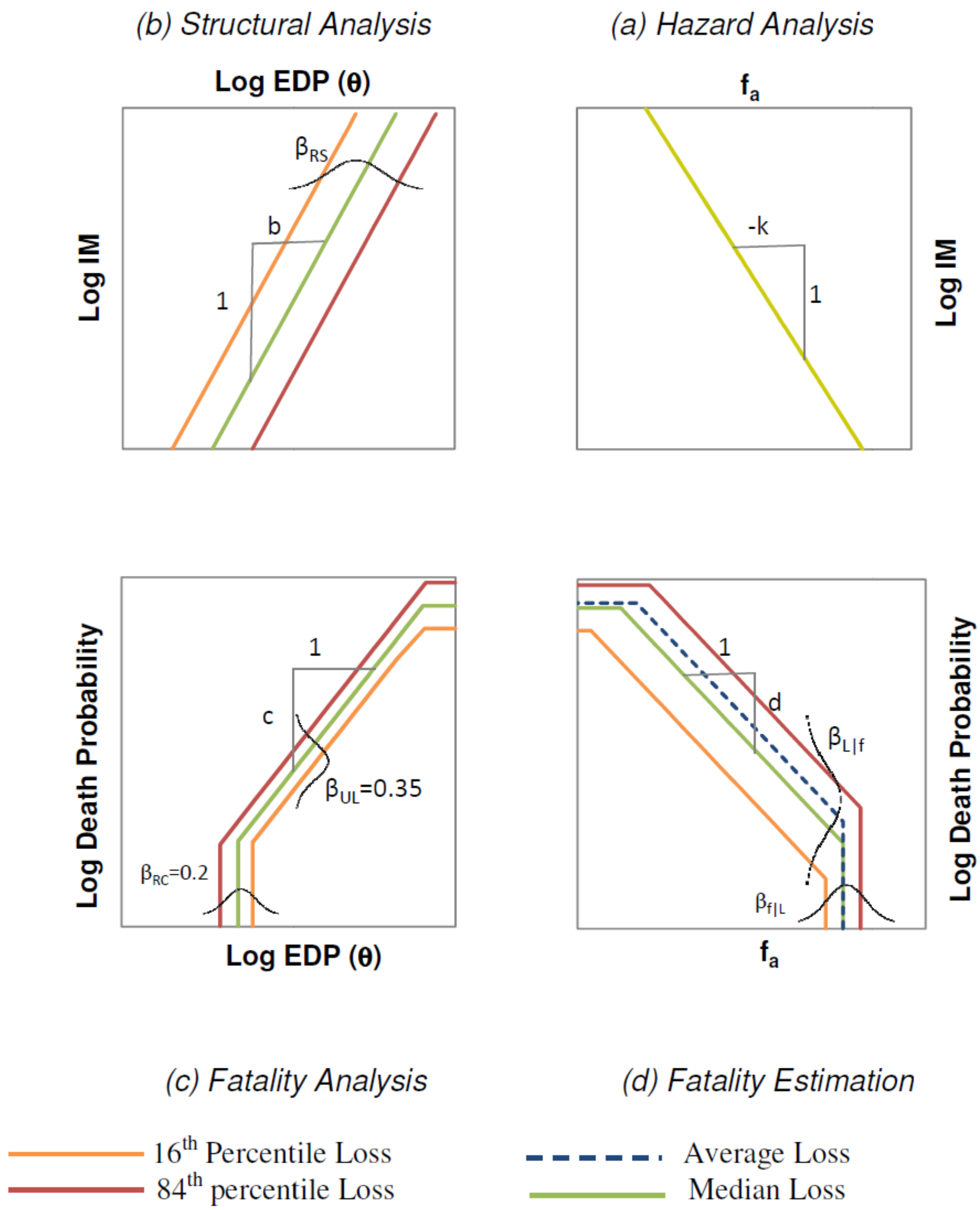


Figure 2.2: Summary of the Four-step approach used to estimate FAR along with various dispersion factors. (a) Two points on hazard recurrence curve are used to compute the IM (hazard analysis). (b) The IM's derived from (a) Are used to compute inter-story drifts using the hazard-drift curve (structural analysis). (c) The drifts obtained from (b) Are used to compute death loss (d) From which FAR is computed.

2.5 Calibration of Death Loss Model

It is evident from Eqs. (2.4) and (2.5) that two parameters needed to calibrate the death loss model, specifically θ_c and c. These parameters are chosen to give a weighted least squares best fit solution to a full analysis resulting from the implementation of Eq. (2.6). Empirically it has been seen that the value of d varies from 0.7 to 0.85 for general case and for special cases, it may exceed 1 (Henley and Kumamoto 1992). So the value of c is calibrated in such a way that d in Eq. (2.2) lies in an expected range.

The restriction on $DL_u \leq 0.75$ (considering occupancy of structure as 75%), and DL_{on} = onset of death loss (when $DL < DL_{on}$, $DL=0$) which is lower bound, can be calculated using:

$$\frac{DL_{on}}{DL_c} = \left|\frac{\theta_{on}}{\theta_c}\right|^c \tag{2.6}$$

where θ_{on} = the onset of damage (normally taken as θ_{on} = θ_{DS2} where θ_{DS2} = drift value for Damage State 2). As the model is calibrated, it is now associated with first two steps of the framework to get the relationship between probability of death loss and frequency.

$$\frac{DL}{DL_{DBE}} = \left|\frac{f_a}{f_{DBE}}\right|^d \tag{2.7}$$

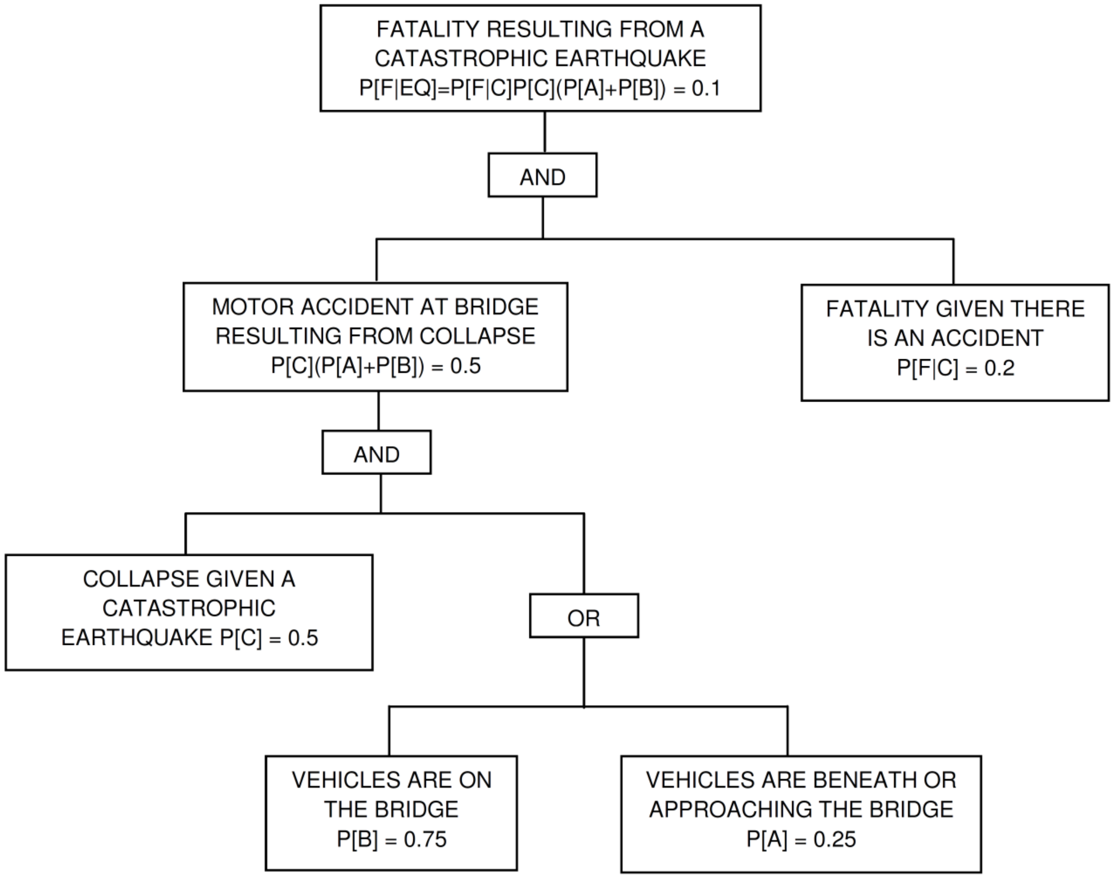


Figure 2.3: Fatal accident probability for a bridge collapse due to a catastrophic earthquake.

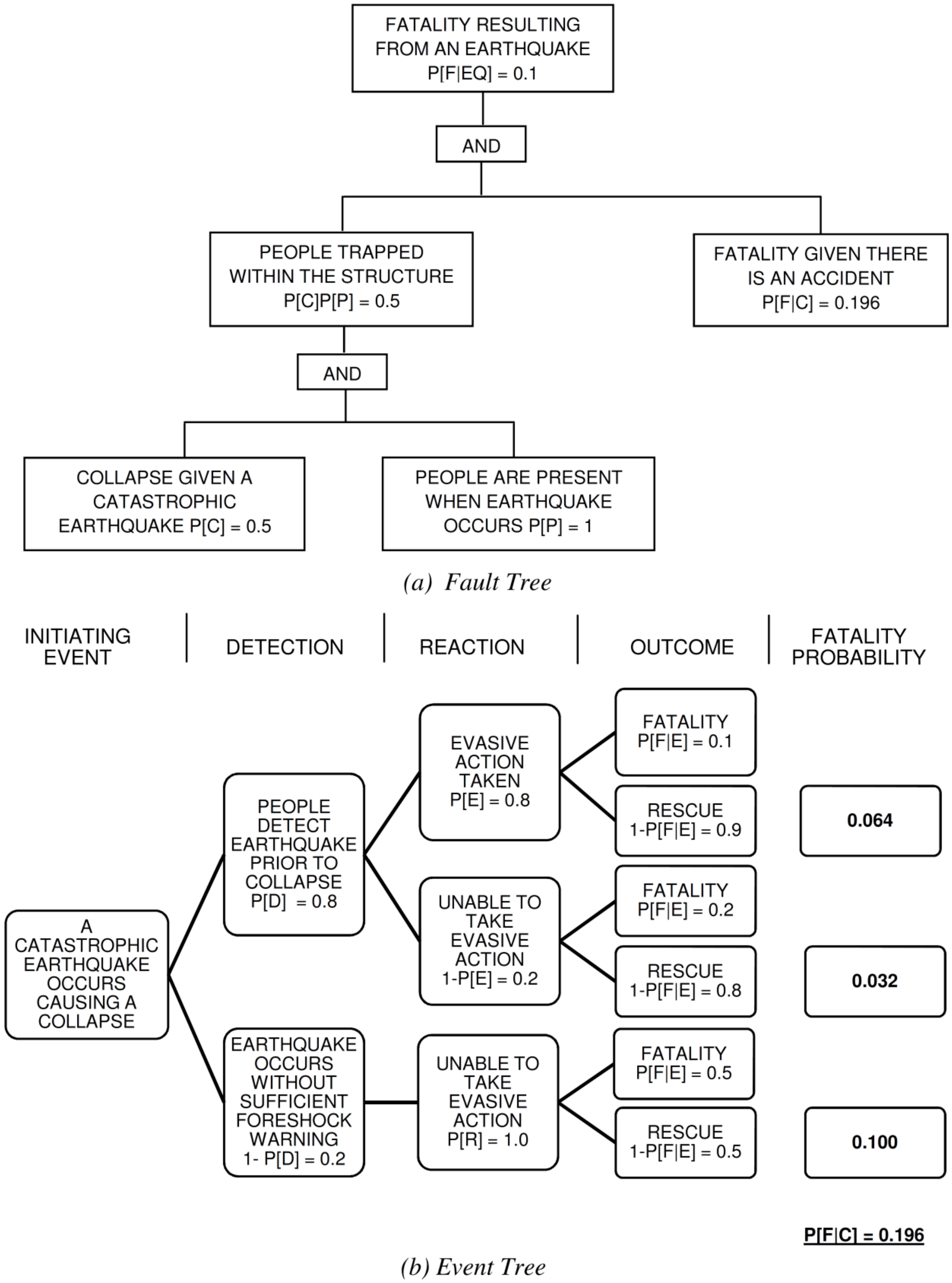


Figure 2.4: Fatal accident probability for a building collapse due to a catastrophic earthquake.

All the equations are probabilistic and values of parameters have an uncertainty and randomness associate with them so it is necessary to incorporate the effects of variability. It involves in both estimating the demand over time produced by the earthquake ground motion and the capacity of structure to resist those demands (Cornell et. al 2002). In this, randomness and uncertainty are considered as aleatoric and epistemic, respectively. Similarly the probability of death loss in Eq. (2.4) contains both aleatoric and epistemic uncertainties. It is essential to transform the median parameters to other fractiles, including the mean values in order to estimate chances of death loss in each steps of the model. By quantifying the kind and degree of uncertainty in each of the parameters concerned, the mean value can be estimated. As the power nature of the death estimation model, a lognormal distribution shall be assumed as appropriate representation of variability. Using the approach outlined by (Kennedy et. al 1980), the total dispersion can be estimated in each of the parameters involved in computing chance of death loss.

Expected Annual Death Loss (EADL) can be calculated by integrating the area under the mean curve of Figure 2.2(d) when that curve is plotted to a natural scale. Thus in integral form EADL may be found by computing following:

$$EADL = \int_0^{f_{on}} DL\, df = DL_u f_u + DL_{DBE} \int_{f_u}^{f_{on}} \left(\frac{f}{f_{DBE}}\right)^d df \qquad (2.8)$$

This can be expressed as:

$$EADL = \left(\frac{\bar{f}_{on}\overline{DL}_{on} + d\bar{f}_u\overline{DL}_u}{1+d}\right) \qquad for\ d \neq -1 \qquad (2.9)$$

$$= \left[\bar{f}_{on}\overline{DL}_{on}\left(1 + \ln\frac{\bar{f}_{on}}{\bar{f}_{u}}\right)\right] \quad for\ d = -1$$

where ($\bar{f}_{on}, \overline{DL}_{on}$) and ($\bar{f}_{u}, \overline{DL}_{u}$), are the mean values of the primary death loss curve coordinates. The mean value of the onset of loss coordinates can be calculated using:

$$\bar{f}_{on} = \tilde{f}_{on} = f_{DBE} \left|\frac{\theta_{DBE}}{\theta_{on}}\right|^{\frac{k}{b}} \tag{2.10}$$

$$\overline{DL}_{on} = \overline{DL}_{DBE}\left|\frac{\bar{f}_{on}}{f_{DBE}}\right|^{d} \tag{2.11}$$

in which $\bar{f}_{on}$, $\tilde{f}_{on}$ are mean and median frequency at onset of Damage State 2; $\overline{DL}_{on}$, $\overline{DL}_{DBE}$ are mean probability of death loss at onset of Damage State 2 and design basis earthquake respectively. $\overline{DL}_{DBE}$ can be estimated using:

$$\overline{DL}_{DBE} = \widetilde{DL}_{DBE}\ \exp\ (0.5\beta_{TL}^{2}) \tag{2.12}$$

$$\widetilde{DL}_{DBE} = \widetilde{DL}_{c}\left|\frac{\theta_{DBE}}{\theta_{c}}\right|^{c} \tag{2.13}$$

$$\beta_{TL}^{2} = \beta_{UL}^{2} + c^{2}(\beta_{RD}^{2} + \beta_{RC}^{2}) \tag{2.14}$$

where β_{RC} and β_{RD} denotes the aleotoric randomness in structural capacity and demand, and β_{UL} represents the epistemic uncertainty in chance of death loss estimation (values are given in Table 2.1); $\widetilde{DL}_{c}$ = median value of probability of death loss at critical drift = 0.1 (Mander and Elms 1994).

Similarly, the coordinates of death losses corresponding to the mean value of complete damage can be computed from:

$$\overline{DL}_u = \widetilde{DL}_u \exp\left(0.5\beta_{UL}^2\right) \tag{2.15}$$

$$\bar{f}_u = f_{DBE}\left|\frac{\overline{DL}_u}{\overline{DL}_{DBE}}\right|^{\frac{1}{d}} \tag{2.16}$$

FAR can be calculated from using *EADL* from Eq. (2.9) by:

$$FAR = 11400\,(EADL) \tag{2.17}$$

where numerical coefficient of 11400 converts the *EADL* into the well known definition of FAR.

2.6 Bridge-Specific Likelihood of Death Loss

Given a catastrophic event, number of people dying can be estimated using the probability of death loss multiplying with the average number of people present (N_p) in the danger zone of length (L+S):

$$N_p = \frac{n\,(AADT)\,(L+S)}{24\,(1000\,V)} \tag{2.18}$$

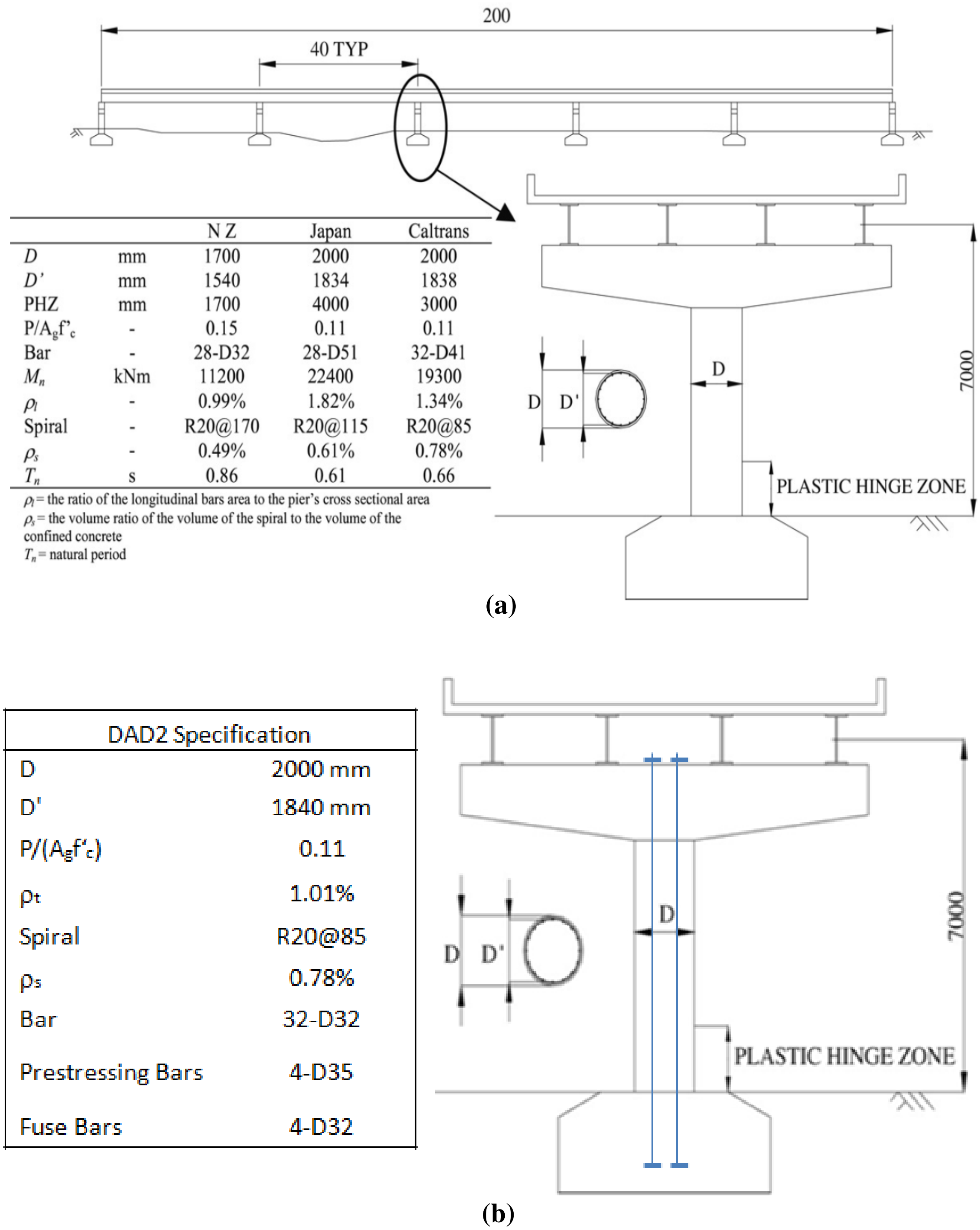
where n = occupancy of vehicle; $AADT$ = annual average daily traffic; L = Length of bridge (m); S = approach stopping distance (m); and V = speed of vehicle (km/h).

2.7 Example Case Studies

The model is implemented on prototype bridges designed for 0.4g ground acceleration with a return period of 10% in 50 years (design basis earthquake, DBE). These bridges are designed for same loading but different specification depending on their location as shown in Figure 2.5(a). These design details of prototype bridges is taken from Mander et al. (2007). DAD bridge is also designed based on California seismicity with same dimension as of Caltrans bridge and referred as DAD2 (Figure 2.5(b)). These prototype bridges have five-spans of 40 m each. The direct loss model is implemented for three different types of structural design as shown Figure 2.6. The seismic design can be considered as ductile design whereas the non-seismic design is regular non-ductile concrete structure details. It is proposed from previous work that DAD uses armored connection details. The combination of rocking action along with post-tensioned prestress tendons and dampers to provide stiffness and supplemental energy dissipation helps in preventing damage. The steel plate at the pier-to-pile cap connection permits the rocking without significant damage to the surrounding concrete (Vamvatsikos and Cornell 2002). The results of direct fatality loss analysis for the different kind of bridge designs are given in Table 2.1 along with different parameters used for the analysis. DAD1 represents damage avoidance design for New Zealand seismicity where as DAD2 is for California seismicity. Figure 2.7 represents the FAR variation for various parameters with the help of swing analysis. This helps in determining the sensitivity of these parameters.

2.8 Discussion

A probabilistic death estimation framework directly relate hazard to response and hence to death. This process works really well by taking into account the probability of death loss due to damage of different earthquake starting from frequent to very rare ones. The aim of the analysis is to consider the humanitarian background while constructing the civil engineering structure. Owners don't want to expose people to more risk just because they are using their facility. The conceptual design of DAD type structures work really well during natural hazards like earthquake but these kinds of structures are still not in use. The model takes into account the different structural strengths and ductility capabilities and the different seismic-hazard frequency relations. These different attributes are all integrated in the evaluation of expected annual death loss. This value helps in calculating FAR, which is good parameter to represent the chance of fatality over a period of 40 years in laymen terms. From this process, indirect losses in monetary terms can be measured, which is a missing from previous studies.

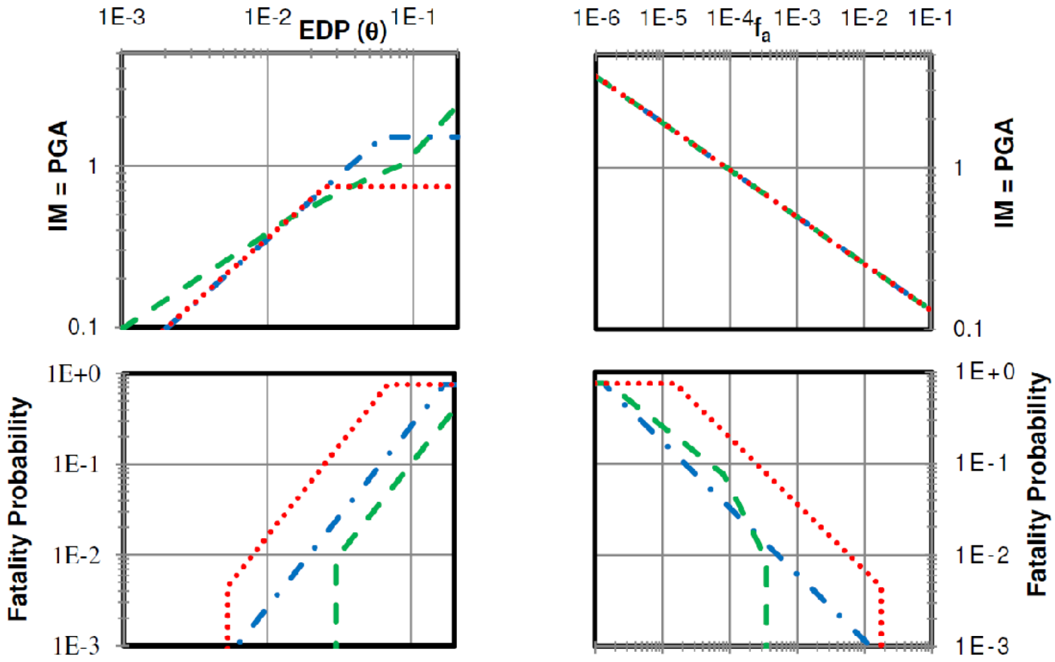


		N Z	Japan	Caltrans
D	mm	1700	2000	2000
D'	mm	1540	1834	1838
PHZ	mm	1700	4000	3000
$P/A_g f'_c$	-	0.15	0.11	0.11
Bar	-	28-D32	28-D51	32-D41
M_n	kNm	11200	22400	19300
ρ_l	-	0.99%	1.82%	1.34%
Spiral	-	R20@170	R20@115	R20@85
ρ_s	-	0.49%	0.61%	0.78%
T_n	s	0.86	0.61	0.66

ρ_l = the ratio of the longitudinal bars area to the pier's cross sectional area
ρ_s = the volume ratio of the volume of the spiral to the volume of the confined concrete
T_n = natural period

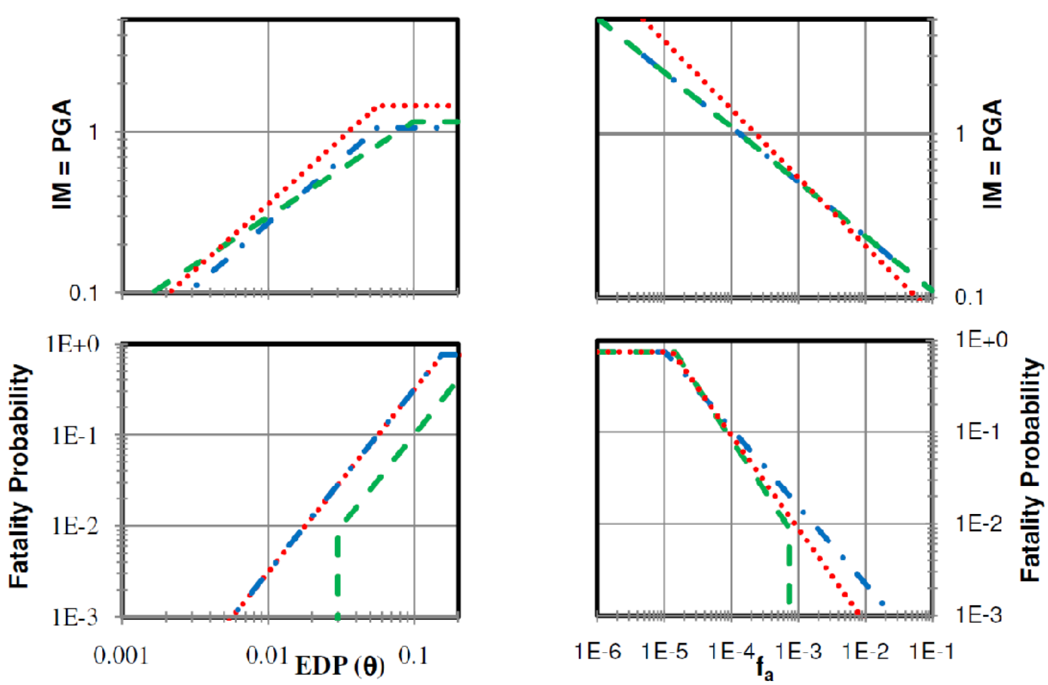
DAD2 Specification	
D	2000 mm
D'	1840 mm
$P/(A_g f'_c)$	0.11
ρ_t	1.01%
Spiral	R20@85
ρ_s	0.78%
Bar	32-D32
Prestressing Bars	4-D35
Fuse Bars	4-D32

Figure 2.5: Five-span prototype bridge used in study. (a) Bridge piers studied by Mander et al. (2007) and (b) DAD pier designed for California seismicity (DAD2).



a) California Bridge for different detailing standards.

CA DAD2 Non-Seismic NZ DAD1 Japan



b) Results for other regions and design approaches.

Figure 2.6: Analysis of fatalities for different bridge designs and regions on log-log scale (DAD1 for NZ and DAD2 is for California seismicity respectively).

Table 2.1: Calculation of expected annual death loss and fatal accident rate (FAR).

	Parameters	Non-Seismic	CALTRANS	DAD2	JAPAN	NZ	DAD1	Remarks
a)	IM_{DBE}	0.4	0.4	0.4	0.4	0.4	0.4	Solberg et al. (2008)
	f_{DBE}	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	
	k	3.45	3.45	3.45	2.4	3	3	
b)	θ_{DBE}	0.0115	0.0117	0.013	0.0115	0.0163	0.0165	IDA Calibration
	b	1.25	1.25	2	1.23	1.27	1.69	
c)	θ_{on}	0.005	0.0053	0.03	0.0053	0.0062	0.03	Sircar et al. (2009)
	θ_c	0.025	0.0616	0.1	0.0566	0.0564	0.1	
	f	1	1	1	1	1	1	
	c	2	2	2	2	2	2	Calibrated
	DL_{DBE}	0.0212	0.0036	0.0017	0.0041	0.0084	0.0027	Eq. (2.13)
Median Parameters	d	-0.725	-0.725	-1.127	-1.025	-0.847	-1.127	Eq. (2.2)
	DL_U	0.75	0.75	0.75	0.75	0.75	0.75	Assigned
	DL_{on}	0.0045	0.0007	0.0090	0.0009	0.0012	0.009	Eq. (2.6)
	f_u	15.3E-06	1.33E-06	1.55E-6	13.1E-6	10.4E-6	14.3E-06	Eq. (2.4)
	f_{on}	0.0178	0.0187	0.0004	0.0095	0.0206	0.00073	
	f_{mid}	-	-	81.4E-06	-	-	-	Solberg et al. (2008)
	DL_{mid}	-	-	0.0755	-	-	-	
Mean Parameters	β_{RD}	0.4	0.42	0.42	0.4	0.43	0.42	IDA Solberg et al. (2008)
	β_{RC}	0.2	0.2	0.2	0.2	0.2	0.2	
	β_{US}	0.25	0.25	0.25	0.25	0.25	0.25	
	β_{UL}	0.35	0.35	0.35	0.35	0.35	0.35	Estimated
	β_{RS}	0.512	0.528	0.528	0.512	0.536	0.528	IDA Solberg et al. (2008)
	$\beta_{Fon\|\theta}$	0.552	0.552	0.345	0.390	0.472	0.355	
	β_{TL}	0.961	0.994	0.994	0.961	1.011	0.994	
	DL_{DBE}	0.0336	0.0059	0.0028	0.0065	0.0139	0.0045	Eq. (2.12)
	DL_U	0.797	0.797	0.797	0.797	0.797	0.797	Eq. (2.15)
	f_u	26.5E-06	2.42E-06	13.8E-6	19.4E-6	17.6E-6	21.1E-6	Eq. (2.16)
	DL_{on}	0.0071	0.0012	0.0194	0.0014	0.0020	0.0148	Eq. (2.10)
	f_{on}	0.0178	0.0187	0.0004	0.0095	0.0206	0.00073	Eq. (2.10)
	DL_{mid}	-	-	0.124	-	-	-	Solberg et al. (2008)
	f_{mid}	-	-	79.2E-06	-	-	-	
	d_{on}	-	-	-1.159	-	-	-	
	d_u	-	-	-0.580	-	-	-	
	EADL	**40.5E-05**	**7.7E-05**	**2.4E-05**	**10.4E-05**	**19.3E-05**	**6.5E-05**	Eq. (2.9)
	FAR	**4.62**	**0.88**	**0.27**	**1.19**	**2.20**	**0.74**	Eq. (2.17)

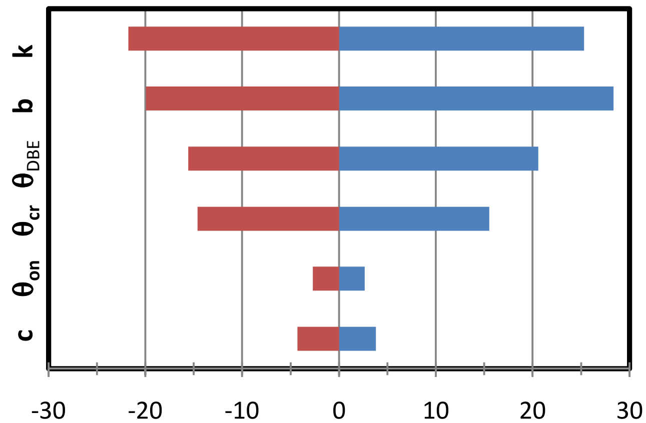


Figure 2.7: Swing analysis with variation of 10% of different parameters for calculation of FAR for DAD1.

2.9 Section Closure

From the research presented in this section the following conclusions are drawn:

1. The results for different kind of structures (Seismic, Non-Seismic and DAD) shows that building structures using DAD technology puts the person on much lesser risk than person sleeping which is socially needed as no-one wants to expose more risk just because he is driving on bridge.
2. For Caltrans and Japan Bridge works as seismically designed ones whereas New Zealand Bridge is can be considered as intermediate designed bridge between seismic and non-seismic.
3. Probability of a collapse will be sensitive to the structures intrinsic ductility capability. Also the risk exposure to individual in a building will depend on where they are located at the time the earthquake strikes and their state of readiness to take evasive action.
4. In this maximum probability of death is considered to be 75% looks more realistic, as building is considered to be occupied fully for 2/3 of the time of the day and remaining time it can be consider as 25% occupied.

3. RAPID LOSS MODELING OF DOWNTIME CAUSED BY SEISMICALLY DAMAGED STRUCTURES

3.1 Summary

Potential design codes and specifications do not include the importance of structures based on downtime losses arising from catastrophic events such as earthquakes. This section seeks to display the importance of downtime in an overall quantitative risk assessment framework. Downtime is referred as the period when a structure is unavailable or it fails to perform to its capacity. The loss model is developed by multiplying the probabilities of being in each of the damage states using vulnerability curves by the corresponding downtime losses and summing those losses across all damage states to give composite downtime with respect to an engineering demand parameter, like drift. The losses are then calibrated to a capped power curve. The calibrated loss model is then incorporated into a direct four-step probabilistic loss modeling framework. This relates seismic hazard to structural response and hence structural response to downtime losses from which scenario losses or the expected annual downtime losses (EADT) for all earthquake hazards are calculated. The downtime losses are the then converted into equivalent monetary losses for bridge-specific examples. The utility of the model is demonstrated for bridges from Caltrans and New Zealand seismicity with different structural detailing. A hypothetical bridge is used to compare 3D losses for bridge design and scenario event.

3.2 Introduction

Natural hazards like earthquakes lead to direct physical damage as well as indirect losses such as death and downtime. Downtime includes the time necessary to plan, finance, and complete repairs of facilities damaged in disaster. Downtime losses can have consequences like delay in reaching the aid to affected people, shut down of necessary business units like food and medical shops etc. Moreover, downtime if lengthy, affects a region's long-term economy. It is important, especially for business organizations, to determine possible downtime as this may lead to much greater losses than just physical damage and death.

The objective of this section is to develop a simplified procedure that relates hazard, to structural response, to downtime and hence estimate losses for key scenario earthquake events, as well as downtime for all hazards leading to expected annual downtime losses (EADL). Generally insurance companies cover losses due to physical damage and death but losses due to downtime may not be covered because compensation is difficult to assess. This can contribute to financial liabilities to individuals/firm and hence to shareholders.

Basoz and Mander (1999) developed fragility curves for assessing the seismic vulnerability of highway bridges through the use of rapid analysis procedures. These curves can be used in various ways as part of a seismic vulnerability analysis methodology for highway bridges. Later, Comerio (2000) estimated the downtime for building structures based on damage states, floor area of building and type of building (wooden, concrete/brick etc.) of U. C. Berkeley campus. The proposed model estimates downtime losses in similar fashion as Mander and Sircar (2009) did for physical damage. They used probability risk assessment methodology to estimate EAL using fragility curve and loss function. After estimating EADL in similar as they did for EAL, the EADL can be associated to monetary losses depending on the context. For example, a building owner will lose rent over the downtime period and while residents will have to pay for shifting and higher rental (price surge) at a new place.

Public assets, such as the highway system are difficult to deal with because ownership and use is collective, thus downtime effects are distributed throughout society at large, but the impact is felt most by the users of those facilities. Private bridge owners will lose toll tax money, whereas public users will have to pay for extra miles and time need to reroute also there will with wear and tear to that route. This will help in comparing 3D losses in monetary terms and their relative significance.

After a catastrophic event, it is essential to get back to normal life as soon as possible. It is desirable for corporations and people want to know the time needed to do so. Delay time can cause bigger losses than direct damage as people may not be able to get necessary aid, either in short or long term; this affects the economy of the region. Often engineers disregard the indirect consequences of structural failure while focusing on the economical minimization of the probability of structural damage and losses. It is important to communicate downtime along with physical damage of structure and the risk of life.

Historically, the importance of a structure is an arbitrary assignment of extra strength by design codes which has been based on engineering judgment and collective experience rather than rigorous analysis.

Mander and Sircar (2009) worked on quantitative risk assessment technique to estimate physical damage losses. For that, a four-step probabilistic approach is used which can be subdivided into four distinct tasks: (a) hazard analysis; (b) structural analysis; (c) damage and hence loss analysis; and (d) loss estimation. Recent research has shown that combination of fragility curves with loss functions can be used for probabilistic risk assessment methodology to estimate expected annual losses for a structure (Kircher et al. 1997; Dhakal and Mander 2006; Mander et al. 2007; Solberg et al. 2008; Sircar et al. 2009). The same procedure is extended herein to calculate the downtime for a given type of structure and earthquake intensity or frequency.

At a constructed facility site, evaluation of seismic hazard and intensity measure (IM) is required for hazard analysis. Structural analysis involves prediction of structural response to increasing levels of ground shaking in terms of engineering demand parameter (EDP). Damage and hence downtime analysis uses EDPs to determine damage measures to the facility components from which downtime can be estimated. Each of these relationships involves uncertainty and must be treated probabilistically from location, seismic demand versus capacity, and capacity versus fatality. The proposed model is calibrated for buildings (Comerio 2000) from U.C. Berkeley campus and bridges of different detailing.

3.3 Proposed Downtime Loss Model

Figure 3.1 presents the four-step loss modeling approach of Mander and Sircar (2009) adapted herein for estimating downtime losses for seismically damaged structures. The main objective of using a direct four-step process for computing losses is to relate estimated losses in terms of well-known seismic demand and structural capacity parameters. These four steps are interrelated through use of log-log graphs from (a) hazard, to (b) response, to (c) damage and (d) hence losses. The relationship between these graphs can be represented using following equation:

$$\frac{DT}{DT_{DBE}} = \left|\frac{\theta}{\theta_{DBE}}\right|^{c} = \left|\frac{S_a}{S_{a\,DBE}}\right|^{bc} = \left|\frac{f_a}{f_{DBE}}\right|^{\frac{bc}{-k}} \tag{3.1}$$

where DT = downtime (weeks) and DT_{DBE} = downtime at design basis earthquake (weeks). f_a = annual frequency; S_a = spectral acceleration; f_{DBE} and $S_{a\ DBE}$ are the annual frequency and spectral acceleration demand (an IM) for design basis earthquake (DBE), typically taken as 10% in 50 years or f_{DBE} = 1/475. θ is the column (or interstory) drift on the structure for the considered event; θ_{DBE} = interstory drift on the structure for design basis earthquake; k = best fit empirical constant for figure 3.1(a); b = exponent that represent slope of the line on log-log plot for figure 3.1(b); c = empirically calibrated power for figure 3.1(c). The slope of the log-log graph in Figure 3.1 (d) is related to first three graphs as:

$$d = \frac{bc}{-k} \tag{3.2}$$

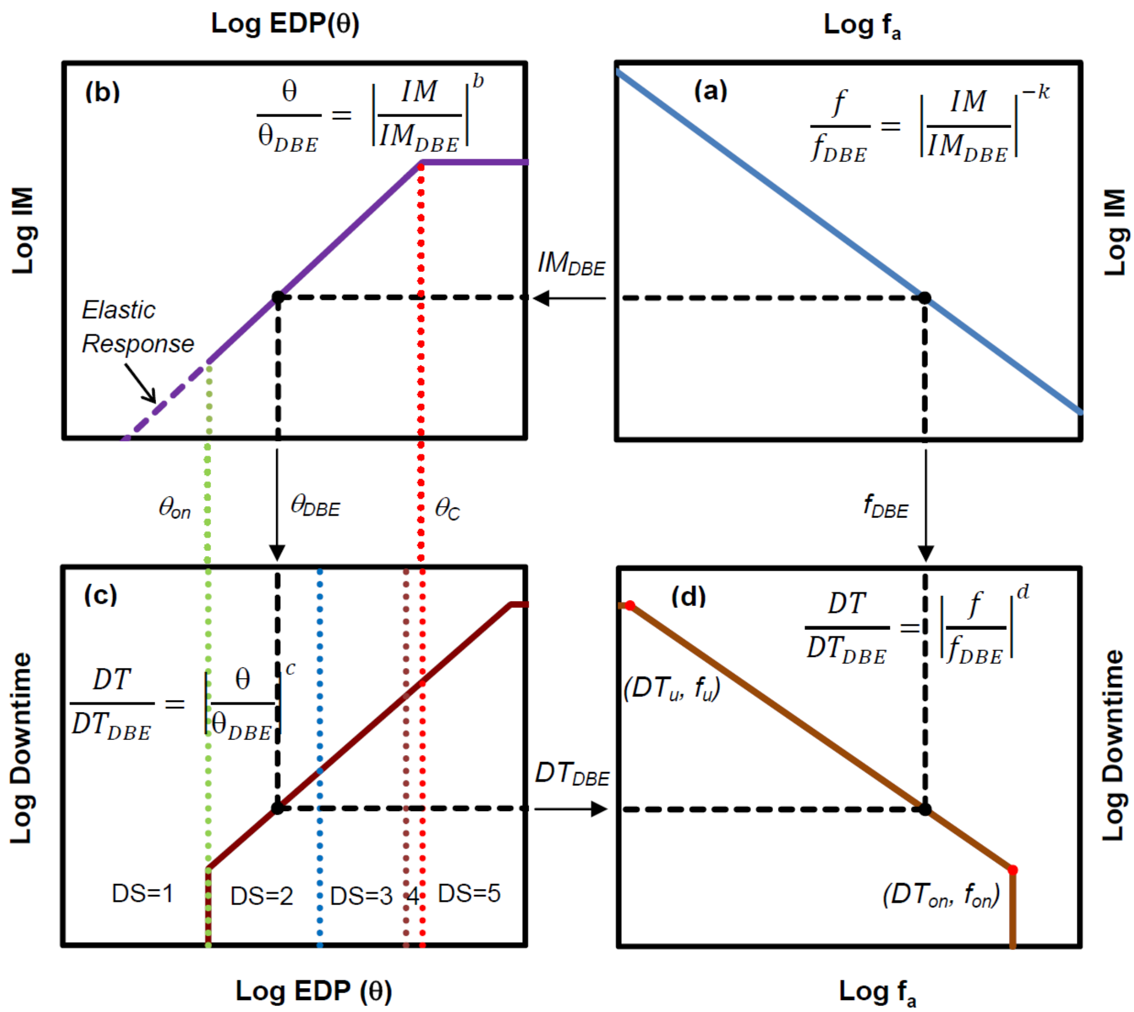


Figure 3.1: Summary of the interrelated four-step approach used to estimate downtime showing, (a) Hazard analysis (b) Structural analysis (c) Downtime analysis and (d) Downtime estimation.

The proposed downtime loss model is a power curve. This also has upper and lower cut-offs; it can be represented using downtime losses in terms of structural drift. This relationship can be expressed as:

$$\frac{DT}{DT_c} = \left|\frac{\theta}{\theta_c}\right|^c \quad ; \quad DT_{on} \le DT \ \le DT_u \tag{3.3}$$

in which DT_c = downtime at onset of damage state 5 (complete damage or toppling); $\theta_c = f\,\theta_{DS5}$ = the critical drift, where θ_{DS5} = drift value for complete damage (collapse), and f = factor to adjust for low damage structures. Generally $f = 1$, but this can have different values for certain special structure types; DT_u = downtime at complete damage or toppling, downtime at this point will be maximum and does not increase after that ($DT \le DT_u = 150$ weeks ~ 3 years, for bridges, Mander and Basoz (1999)); DT_{on} = lower bound on downtime is based on the concept that there will be no damage to structure when earthquake intensity is less than damage state 2 (when $DT < DT_{on}$, DT =0) and can be estimated as:

$$\frac{DT_{on}}{DT_c} = \left|\frac{\theta_{on}}{\theta_c}\right|^c \tag{3.4}$$

where θ_{on} = the onset of damage (normally taken as $\theta_{on} = \theta_{DS2}$ = drift value for Damage State 2), at this state the structure is expected at least some kind of inspection by experts which can lead to downtime.

It is evident from Eqs. (3.3) and (3.4) that several parameters need to be calibrated for the loss model. These parameters are chosen to give a weighted least squares best-fit solution to a full analysis resulting from implementation of Eq. (3.3). Mander and Sircar (2009) used vulnerability curves along with losses to derive the total probable loss. A similar approach is used herein this leads to the following equation. The total probable downtime loss due to earthquake of a given probability is the sum of the corresponding value for the damage states and it can be expressed as:

$$DT[EDP] = \sum_{i=2}^{5} P_i[EDP]DT_i \tag{3.5}$$

where $P_i[EDP]$ and DT_i are the respective probability and downtime loss for ith damage state.

3.4 Calibration of Loss Model for Downtime

Mander and Basoz (1999) postulated the downtime shown in Table 3.1 for each of the 5 HAZUS damage states for highway bridges.

Table 3.1: Definition of damage states and performance outcomes (Mander and Basoz 1999).

Damage State	Descriptor for Degree of Damage	Post-earthquake Utility of Structure	Repairs Required	Outage Expected
1	None (pre-yield)	Normal	None	-
2	Minor/slight	Slight damage	Inspect, adjust, patch	< 3 days
3	Moderate	Repairable damage	Repair components	< 3 weeks
4	Major/extensive	Irreparable damage	Rebuild components	< 3 months
5	Complete/collapse	Irreparable damage	Rebuild structure	> 3 months

Table 3.1 is then used to multiply the vulnerability probability with expected losses (consequences) to obtain probable losses for a given EDP. The sum of the product of probability of drift and its expected outcome (Eq. (3.5)) is then calibrated using the power curve with the upper and lower cutoffs. The model is calibrated for different bridges located in California and New Zealand. For each type of bridge, the sum of product of downtime losses from Table 3.1 and the lognormal distributed drift corresponding to each damage state (Solberg et al. 2008; Sircar et al. 2009) is calculated and a curve is generated. The median values thus obtained are curve fitted using power curve with upper and lower cutoff as shown in Figure 3.2. It is observed from the results that the exponent of calibrated downtime loss model, '*c*', for bridges is 2.5 (Figure 3.2). Similary, a relationship is established between EDP and downtime losses for buildings using downtime loss data from Comerio (2000). The critical drift for buildings is assumed as 0.06 (Kircher et al. 1997). For buildings, the downtime at critical drift depends on floor area and the exponent '*c*' decreases with increase in floor area (Figure 3.3). There are considerable epistemic uncertainties in these estimates due to contractual variabilities and scope of work, this is discussed below.

3.5 Uncertainty and the Analysis of EADT

Using the calibrated downtime loss model along with hazard and structural analysis, downtime loss estimation can be developed. Utilizing dispersion factors, in conjunction with median coordinates, an expected value (mean) loss curve can be developed. Expected annual downtime (EADT) may be estimated by simply calculating the area under mean curve of the Figure 3.1(d) which can expressed as:

$$EADT = \left(\frac{\bar{f}_{on}\overline{DT}_{on} + \ d\bar{f}_u\overline{DT}_u}{1+d}\right) \qquad for\ d\ \neq -1 \tag{3.6}$$

where ($\bar{f}_{on}, \overline{DT}_{on}$) and ($\bar{f}_u, \overline{DT}_u$), are the mean values of the primary downtime loss curve coordinates.

The mean values of coordinates can be estimated using:

$$\bar{f}_{on} = \tilde{f}_{on} = f_{DBE}\left|\frac{\theta_{DBE}}{\theta_{on}}\right|^{\frac{k}{b}} \tag{3.7}$$

where $\bar{f}_{on}$ and $\tilde{f}_{on}$ are mean and median value of the frequency at the onset of damage, respectively. Note these are identical because the underlying distribution of damage onset is assumed to be normal. Similarly using Eq. 3.1 in terms of mean parameters:

$$\overline{DT}_{on} = \overline{DT}_{DBE}\left|\frac{\bar{f}_{on}}{f_{DBE}}\right|^{d} \tag{3.8}$$

in which:

$$\overline{DT}_{DBE} = \widetilde{DT}_{DBE}\ \exp\left(0.5\beta_{TL}^2\right) \tag{3.9}$$

where

$$\widetilde{DT}_{DBE} = \widetilde{DT}_c\left|\frac{\theta_{DBE}}{\theta_c}\right|^{c} \tag{3.10}$$

and

$$\beta_{TL}^2 = \ \beta_{UL}^2 + c^2(\beta_{RD}^2 + \beta_{RC}^2) \tag{3.11}$$

where $\widetilde{DT}_c$ = median downtime at critical drift; β_{UL} = epistemic uncertainty in downtime loss estimation = 0.35; with β_{RD} and β_{RC} being the aleotoric randomness dispersions in structural capacity and demand, respectively.

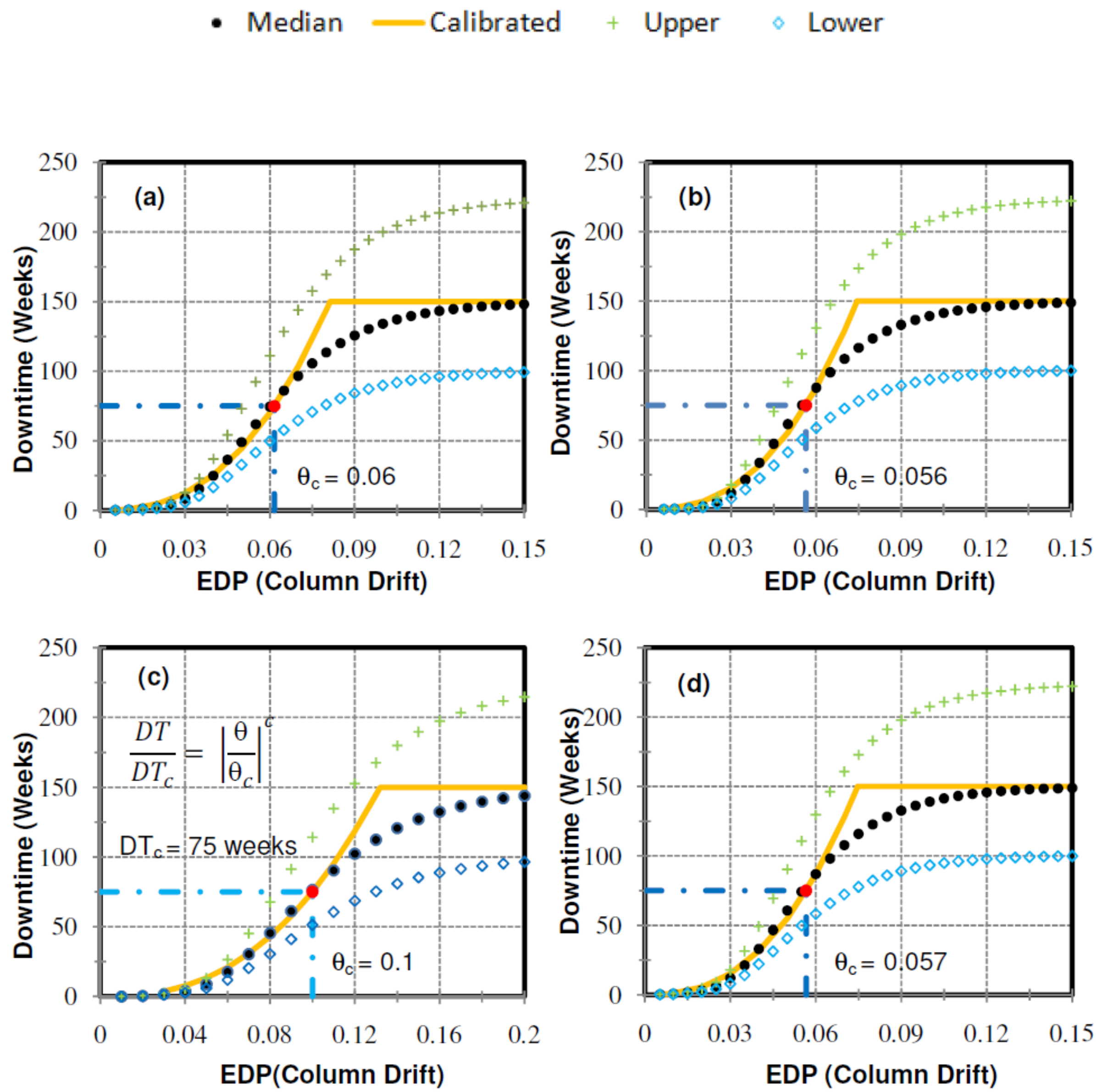


Figure 3.2: Loss model calibration for post-earthquake downtime losses. These curves are plotted showing upper, median and lower ranges of loss based on Eq. 3.5 for (a) NZ; (b) Caltrans; (c) DAD1; and (d) Japan. The solid line shows the capped power model according to Eq. 3.4. For this case the onset of Damage State 5 is shown as the critical drift point, θ_c. All calibrated curves have the same power of c = 2.5.

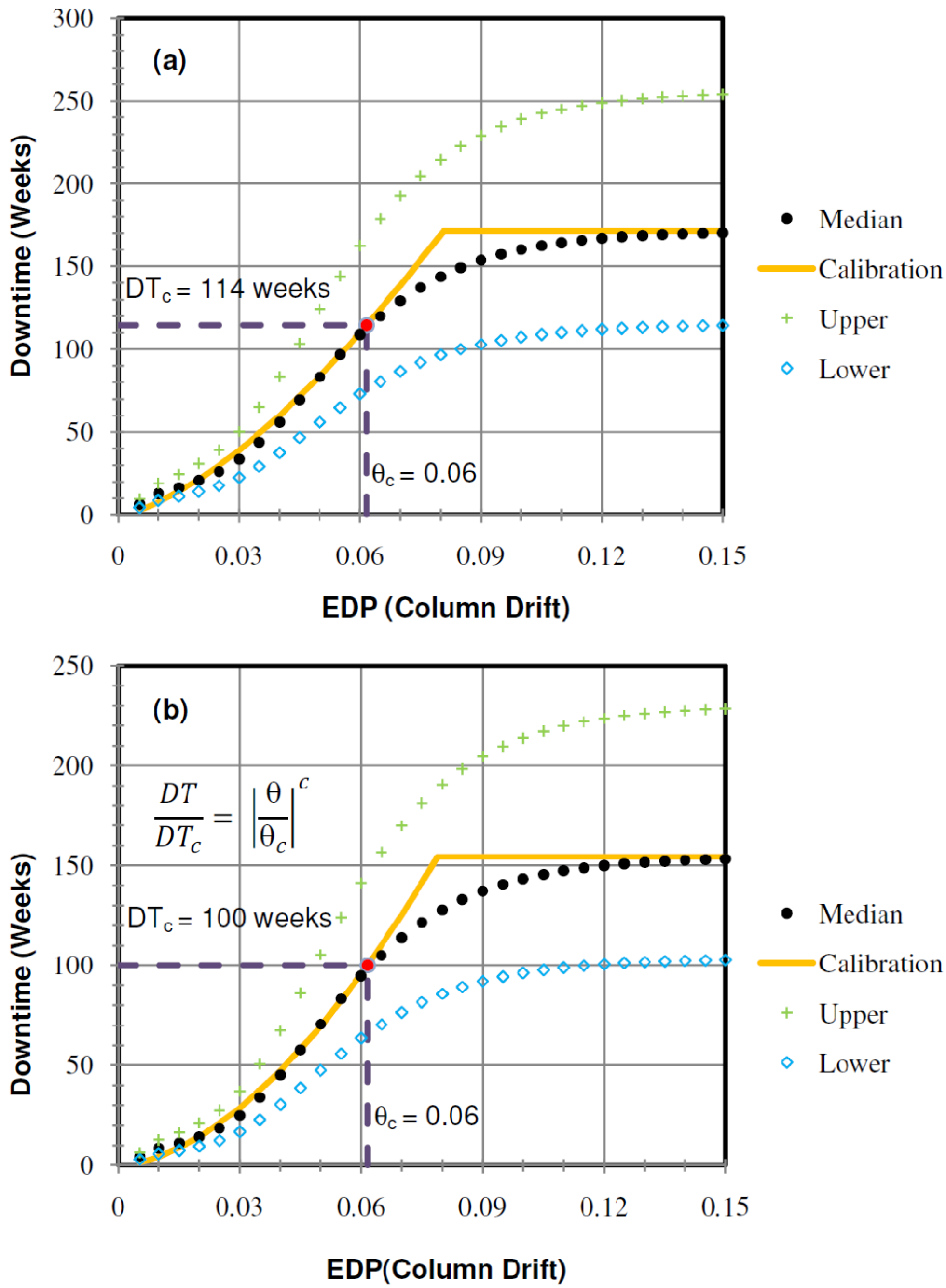


Figure 3.3: Downtime for given engineering demand parameter (column drift) using data from Comerio (2000) for: (a) Large structural buildings with floor area more than 80,000 square feet (c = 1.5); and (b) Small structural buildings with floor area less than 80,000 square feet (c = 1.75). θ_c = 0.06 is a typical complete damage drift taken for buildings (Kircher et al. 1997).

The mean downtime loss at complete collapse or toppling is:

$$\overline{DT}_u = \widetilde{DT}_u \exp\left(0.5\beta_{UL}^2\right) \tag{3.12}$$

$$\bar{f}_u = f_{DBE}\left|\frac{\overline{DT}_u}{\overline{DT}_{DBE}}\right|^{\frac{1}{d}} \tag{3.13}$$

Values of these aleatoric and epistemic uncertainties are given in Table 3.2 along with the associated dispersion factors.

3.6 Case Study for Bridges

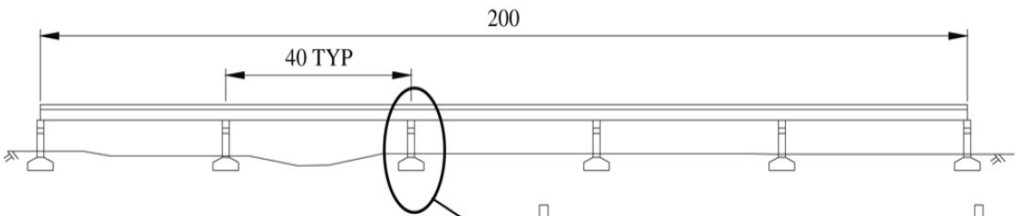
The prototype of bridge for Caltrans, New Zealand and Japan is shown in Figure 3.4(a). The DAD design for California seismicity with same dimension as of Caltrans bridge is shown in Figure 3.4(b). Four-step model for California seismicity is shown in Figure 3.5 for non-seismic, Caltrans and DAD2 bridges whereas comparison of bridge designs from New Zealand, Japan and DAD1 (designed for NZ seismicity) is shown in Figure 3.6. The results of four-step model for various bridges are tabulate in Table 3.2 with mean and median parameters.

Result in Table 3.2 shows that EADT for non-seismically designed bridges is 2.11 days whereas for bridge designed using Caltrans specifications has only 6 hours of expected annual downtime. Bridge designed using DAD technique in California seismicity (DAD2) performs better than Caltrans design with only 3.6 hours as expected annual downtime. Bridges designed to New Zealand specifications show more than 16.5 hours as downtime and DAD designed in NZ seismicity (DAD1) has only 5.5 hours as expected annual downtime.

The prototype bridge is considered to see the relative importance of 3D losses and compare bridge losses on monetary scale. The prototype bridge of five-spans 40 m each and 15 m width is assumed to be constructed with a cost of \$1,200/m^2. The annual average daily traffic (AADT) is assumed as 60,000 with occupancy of a vehicle as 1.2. Based on U.S. Department of Transportation (2007) recommendations, the value of statistical life is assumed as \$ 6.0 million. The toll tax is assumed as the sum of amount the bridge owner will lose as toll, the cost of extra miles traveled by the driver to reroute and wear and tear to the rerouted route. It is conservatively assumed as \$1.0 per vehicle per pass. Result shows that Caltrans design saves more than \$149,000 per year by just adapting the ductile design and DAD design can save another \$11,000 per year just by improving the detailing of connections (Table 3.3). Table 3.3 shows 3D losses for different prototype bridges in terms of total expected annual losses.

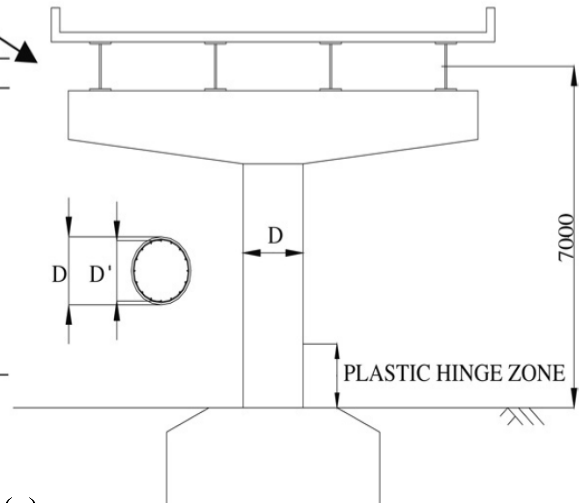
It is also of interest to have some insight as to the expected downtime for the two commonly used earthquakes applied in design, specifically DBE and MCE (maximum considered earthquake, 2% in 50 years). Table 3.4 shows the scenario losses for DBE and MCE in California region based on different detailing. Table 3.5 uses these scenario losses from Table 3.4 for prototype bridge and estimate losses in monetary terms.

Figure 3.7 represent the total expected annual losses in stacked column chart for all these different prototype bridges and Figure 3.8 represent scenario event losses for bridges in Caltrans seismicity showing individual Damage, Death and Downtime losses.



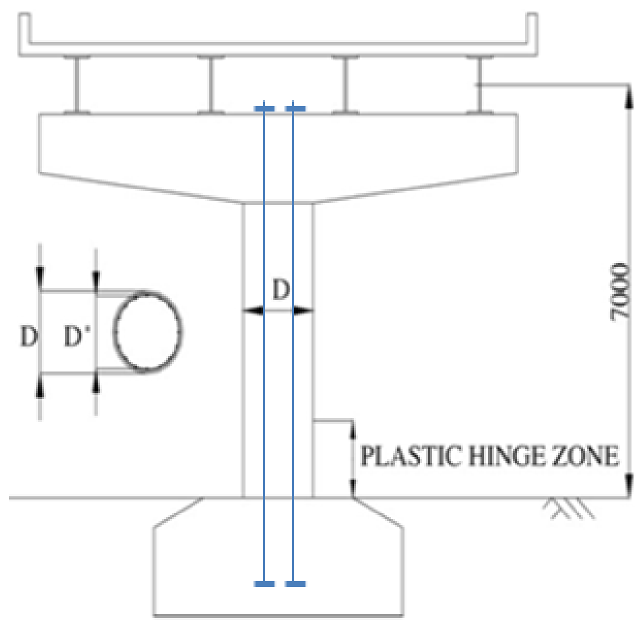
		N Z	Japan	Caltrans
D	mm	1700	2000	2000
D'	mm	1540	1834	1838
PHZ	mm	1700	4000	3000
$P/A_g f'_c$	-	0.15	0.11	0.11
Bar	-	28-D32	28-D51	32-D41
M_n	kNm	11200	22400	19300
ρ_l	-	0.99%	1.82%	1.34%
Spiral	-	R20@170	R20@115	R20@85
ρ_s	-	0.49%	0.61%	0.78%
T_n	s	0.86	0.61	0.66

ρ_l = the ratio of the longitudinal bars area to the pier's cross sectional area
ρ_s = the volume ratio of the volume of the spiral to the volume of the confined concrete
T_n = natural period



(a)

DAD2 Specification	
D	2000 mm
D'	1840 mm
$P/(A_g f'_c)$	0.11
ρ_t	1.01%
Spiral	R20@85
ρ_s	0.78%
Bar	32-D32
Prestressing Bars	4-D35
Fuse Bars	4-D32



(b)

Figure 3.4: Five-span prototype bridge used in study. (a) Bridge piers studied by Mander et al. (2007) and (b) DAD pier designed for California seismicity (DAD2).

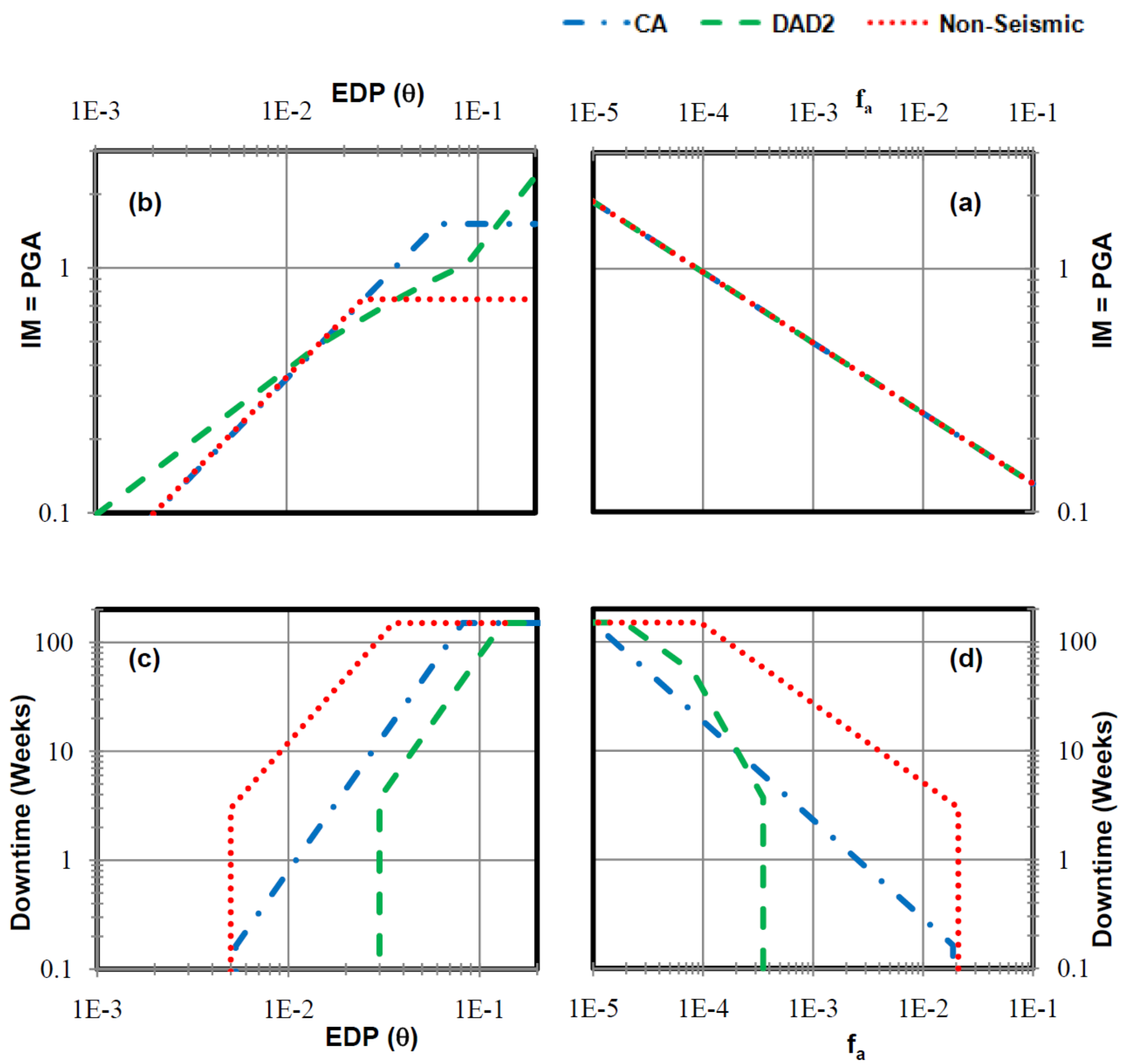


Figure 3.5: Comparison of median values of downtime loss on log-log scale for three different design systems Caltrans, DAD2 and Non-Seismic using four-step approach.

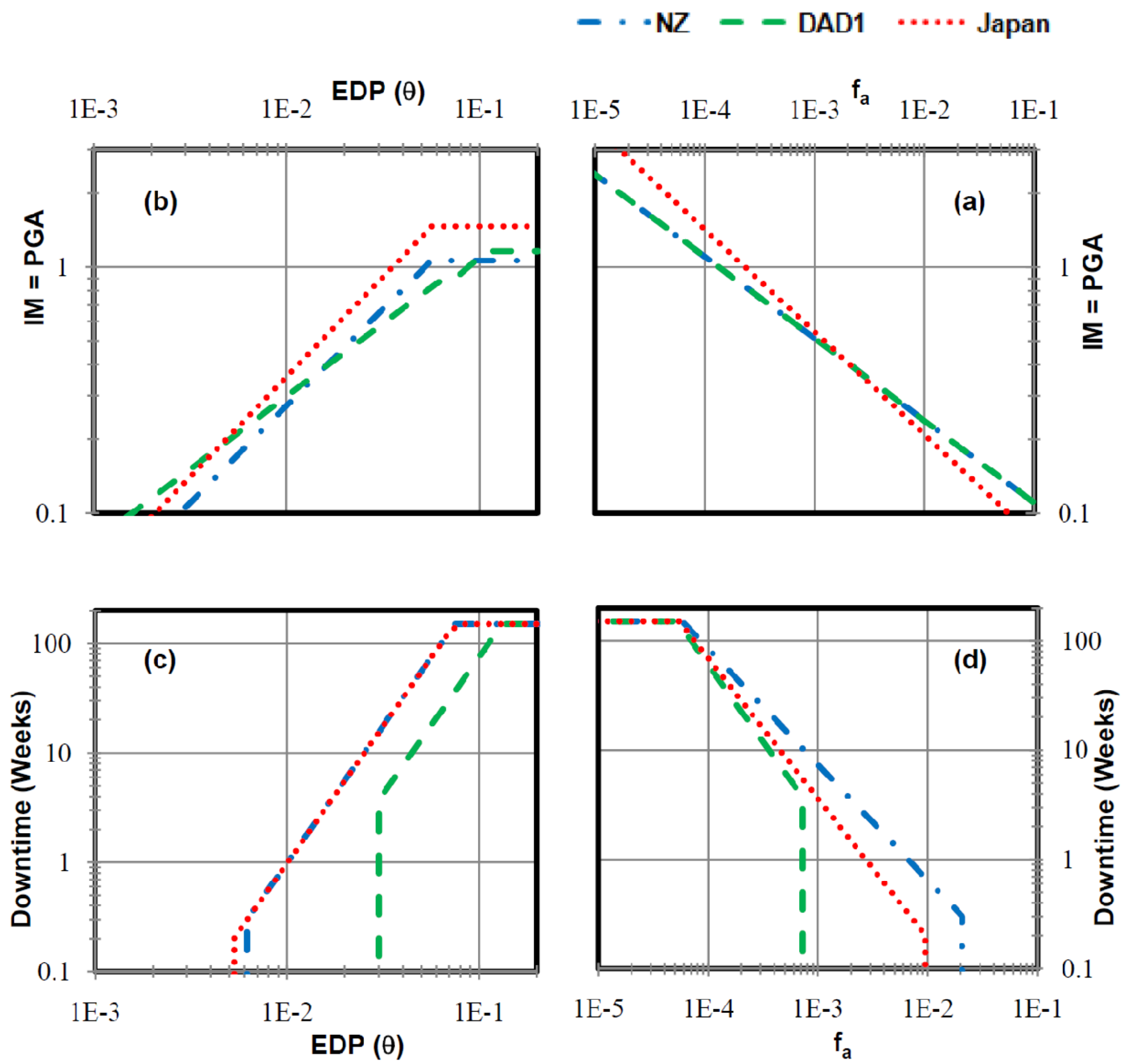


Figure 3.6: Comparison of median values of downtime loss on log-log scale for three different design systems New Zealand, DAD1 and Japan using four-step approach.

Table 3.2: Calculation of expected annual downtime (in Days) for different bridge designs along with median and mean parameters.

	Parameters	Non-Seismic	CALTRANS	DAD2	JAPAN	NZ	DAD1	Remarks
a)	IM_{DBE}	0.4	0.4	0.4	0.4	0.4	0.4	Solberg et al. (2008)
	f_{DBE}	0.0021	0.0021	0.0021	0.0021	0.0021	0.0021	
	k	3.45	3.45	3.45	2.4	3	3	
b)	θ_{DBE}	0.0115	0.0117	0.013	0.0115	0.0163	0.0165	IDA Calibration
	b	1.25	1.25	2	1.23	1.27	1.69	
c)	θ_{on}	0.005	0.0053	0.03	0.0053	0.0062	0.03	Sircar et al. (2009)
	θ_c	0.025	0.0616	0.1	0.0566	0.0564	0.1	
	f	1	1	1	1	1	1	
	c	2	2.5	2.5	2.5	2.5	2.5	Calibrated
	DT_{DBE}	15.87	1.179	0.457	1.396	3.368	0.8294	Eq. 3.10
Median Parameters	d	-0.725	-0.906	-1.449	-1.281	-1.058	-1.408	= b c / -k
	DT_U	150	150	150	150	150	150	Assigned
	DT_{on}	3	0.163	3.697	0.201	0.3	3.697	Eq. 3.4
	f_u	9.46E-05	9.97E-06	1.93E-05	5.5E-05	5.81E-05	5.24E-05	Eq. 3.3
	f_{on}	0.02092	0.01868	0.00035	0.0095	0.0206	0.0007	Eq. 3.7
	f_{mid}	-	-	8.14E-05	-	-	-	Solberg et al. (2008)
	DT_{mid}	-	-	52.8	-	-	-	
Mean Parameters	β_{RD}	0.42	0.42	0.42	0.4	0.43	0.42	IDA Solberg et al. (2008)
	β_{RC}	0.2	0.2	0.2	0.2	0.2	0.2	
	β_{US}	0.25	0.25	0.25	0.25	0.25	0.25	
	β_{UL}	0.35	0.35	0.35	0.35	0.35	0.35	Estimated
	β_{RS}	0.5281	0.5281	0.5281	0.5123	0.5361	0.5281	Calculated
	$\beta_{Fon\mid\theta}$	0.552	0.552	0.345	0.390	0.472	0.355	= $k/b*\beta_{RC}$
	β_{TL}	0.994	1.215	1.215	1.172	1.236	1.215	Eq. 3.11
	DT_{DBE}	26.01	2.465	0.956	2.77	7.23	1.73	Eq. 3.9
	DT_U	159.47	159.47	159.47	159.47	159.47	159.47	Eq. 3.12
	f_u	17.2E-05	2.10E-05	6.15E-05	8.9E-05	11.3E-05	8.47E-05	Eq. 3.13
	DT_{on}	4.92	0.34	12.75	0.40	0.645	7.73	Eq. 3.8
	f_{on}	0.0209	0.0187	0.0004	0.0095	0.0206	0.0007	Eq. 3.7
	DT_{mid}	-	-	110.38	-	-	-	Solberg et al. (2008)
	f_{mid}	-	-	7.92E-05	-	-	-	
	d_{on}	-	-	-1.449	-	-	-	
	d_u	-	-	-0.725	-	-	-	
	EADT (days)	**2.11**	**0.25**	**0.15**	**0.36**	**0.69**	**0.23**	Eq. 3.6

Table 3.3: Comparative study of a prototype bridge showing total loss (Damage, Death and Downtime) per year for different bridge designs.

Parameters	Non Seismic	Caltrans	DAD2	Japan	NZ	DAD1
EAL ($/million)	6,210	917	176	970	1,773	328
FAR	4.62	0.88	0.27	1.19	2.20	0.74
EADL	0.000405	7.72E-05	2.37E-05	0.000104	0.000193	6.49E-05
EADT	2.11	0.25	0.15	0.36	0.69	0.23
Damage Loss	22,356	3,301	634	3,492	6,383	1,181
Human Loss	22,664	4,317	1,325	5,838	10,792	3,630
Downtime Loss ($)	126,600	15,000	9,000	21,600	41,400	13,800
Total Loss ($)/year	**171,620**	**22,618**	**10,958**	**30,930**	**58,575**	**18,611**
% Cost of Bridge	**4.77**	**0.63**	**0.30**	**0.86**	**1.63**	**0.52**

Analysis Assumptions
Length of bridge = 200m (five spans of 40m); Breath = 15m;
Construction Cost = $1,200/m^2 ; Total Construction Cost = $ 3.6 million
Vehicle Speed = 100km/h; Vehicle Occupancy = 1.2
Annual Average Daily Traffic, AADT = 60,000
Value of a Statistical Life, VSL = $ 6 million
Approach stopping distance =100m (for 100 km/h speed)
Assumed equivalent toll tax for each passage = $ 1.0

Table 3.4: Comparative study of scenario results for DBE and MCE in California seismicity.

Scenario Results (Median Values)			
	Non-Seismic	**Seismic**	**DAD2**
Death Probability			
DBE 10% in 50 year EQ	0.023	0.004	-
MCE 2% in 50 year EQ	0.07	0.013	-
Downtime (Weeks)			
DBE 10% in 50 year EQ	17	1.3	-
MCE 2% in 50 year EQ	52	5.4	-
Damage (Loss ratio)			
DBE 10% in 50 year EQ	0.38	0.051	-
MCE 2% in 50 year EQ	0.8	0.16	-

Table 3.5: 3D losses for prototype bridge in monetary terms for DBE and MCE scenarios.

		Non-Seismic	Relative damage	Seismic	Relative damage
DBE	Damage	$1,368,000	1	$183,600	1
	Death	$1,490,400	1.1	$259,200	1.4
	Downtime	$7,140,000	5.2	$546,000	3.0
	Total	**$9,998,400**		**$988,800**	
MCE	Damage	$2,880,000	1	$576,000	1
	Death	$4,536,000	1.6	$842,400	1.5
	Downtime	$21,840,000	7.6	$2,268,000	3.9
	Total	**$29,256,000**		**$3,686,400**	

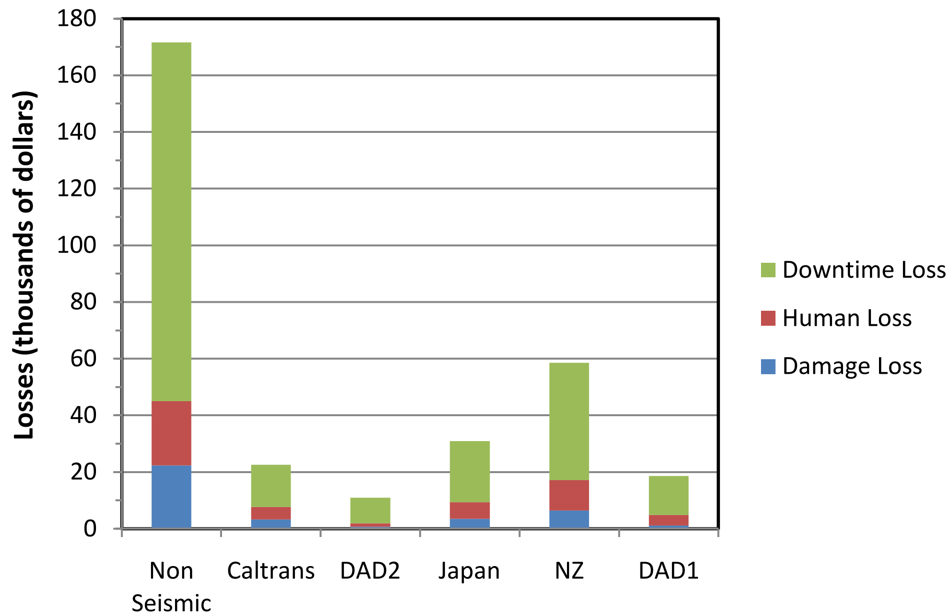


Figure 3.7: Comparative study of expected annual 3D losses of various prototype bridges.

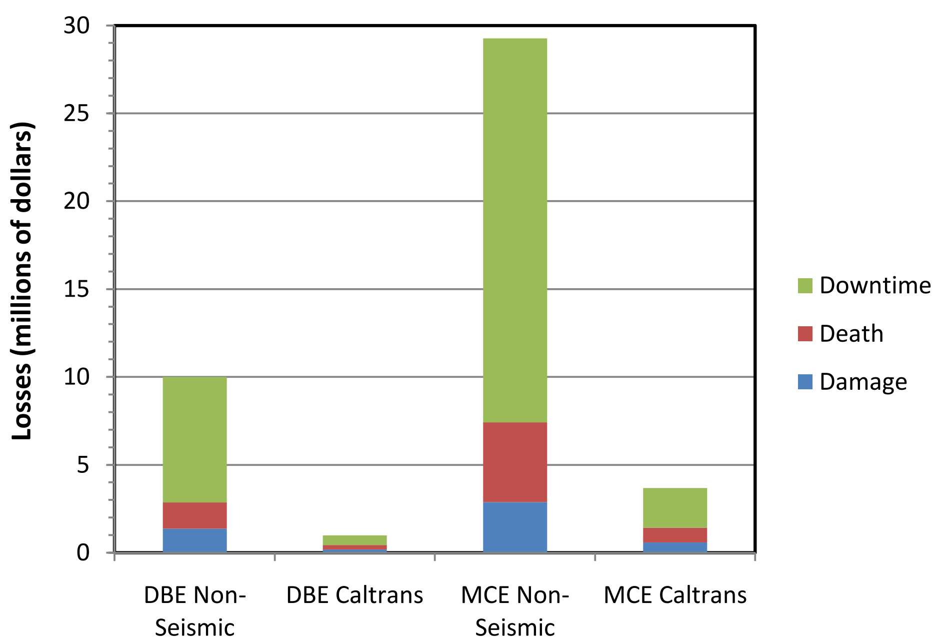


Figure 3.8: Comparative study of 3D losses for scenario event (DBE and MCE) for Non-Seismic and Caltrans prototype bridges.

3.7 Discussion

Although the simple direct four-step probabilistic loss estimation framework works well in estimating the 3D losses. The absolute accuracy may be quite dependent on certain parameters and assumptions leaving the final results in doubt. Notwithstanding this, for comparative purposes the results should remain consistent amongst the different options considered. As it can be seen from the results, downtime losses are the most significant for the cases investigated. Results shows that compared to their historic non-seismic counterparts, modern ductile bridges designed in accordance with contemporary seismic design specifications (e.g. Caltrans) have significantly less total losses expected. Improved detailing using for example the emerging DAD armoring details further improves the overall system performance. The cost for making any of these improvement changes in the design is negligible, and therefore clearly worthwhile. Also, when comparing the two DAD designs, DAD1 vs. DAD2, the latter is essentially the same, but stronger – a clear benefit.

A sensitivity analysis was carried out to investigate the effects of key parameters on downtime losses. The parameters were varied (one at a time) by 10% and corresponding change in downtime losses measured (Figure 3.9). It is observed that the calibrated loss model has least effect on the estimated downtime, as 10% variation in c can affect less than 2% the estimated downtime but is most sensitive to the use parameter, AADT and tax. Similarly, Figure 3.10 considers the swing analysis in total expected annual losses for various parameters of prototype DAD2 bridge.

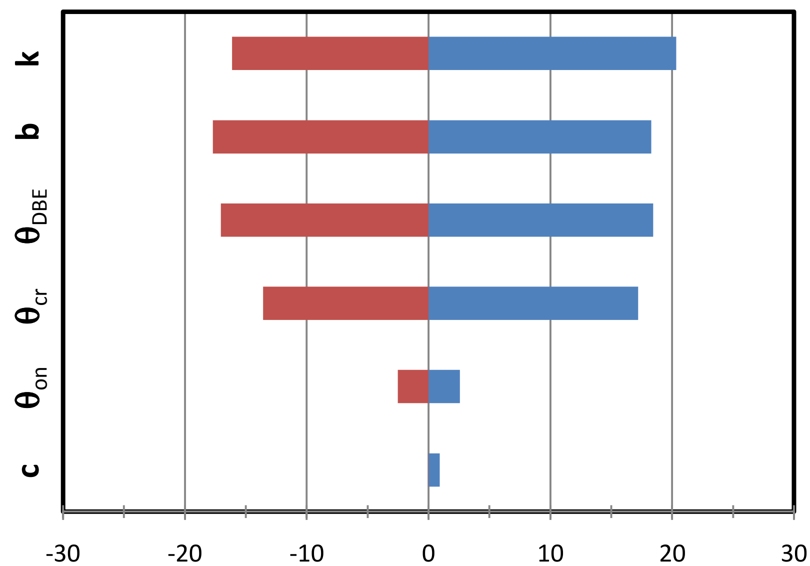


Figure 3.9: Swing analysis with variation of 10% of different parameters for calculation of expected annual downtime losses for DAD1.

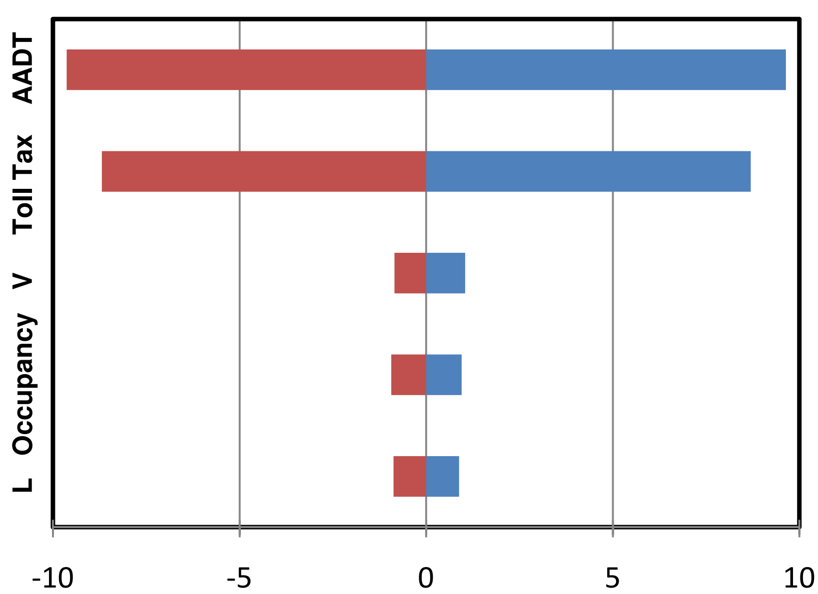


Figure 3.10: Swing analysis with variation of 10% of different parameters for calculation of total expected annual losses for prototype DAD2 bridge.

3.8 Section Closure

From the research presented in this section the following conclusions are drawn:

1. The four-step probabilistic rapid loss estimation framework can be easily applied to estimate 3D losses and can be calibrated for different kinds of simple bridge structures.
2. Downtime leads to the most significant losses which can totally overshadow the other losses due to damage and death. Clearly, it is really important to consider downtime losses for constructed facilities in public ownership as a significant sector of society become affected when such facilities are out of commission. Likewise, similar constructed facilities in private sector ownership are also affected. Lack of business continuity due to significant downtime may affect long term business viability.
3. The emerging DAD approach can also reduce losses both in overall EAL, but perhaps more significantly losses under DBE and MCE can essentially be eliminated with judicious design.
4. Ductile design reduces 3D losses significantly for smaller earthquakes and it fails to perform in larger once. Although ductility clearly helps, there is also no substitution for high strength.

4. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

4.1 Summary

It is important to communicate the risk to life facility downtime associated with the physical damage to structures arising from natural hazards. A quantitative risk assessment technique was used to examine the total losses due to earthquakes. This work extended the four-step direct loss modeling developed by Mander and Sircar (2009). That approach used a combination of fragility curves and loss functions used for probabilistic risk assessment methodology to estimate expected annual loss for structure. The four-step approach was subdivided into four distinct tasks ((a) Hazard analysis, (b) Structural analysis, (c) Loss analysis and (d) Loss estimation) and was conducted for damage, death and downtime (the 3D's) for common class of highway bridges.

The expected annual loss was derived from all possible earthquakes after accounting for the uncertainties in the response and modeling process. Empirically calibrated models in the form of power curves that are capped with upper and lower bounds were used to model death and downtime losses from the onset of damage until toppling or collapse occurs.

The utility of the approach was investigated for the bridges in both California and New Zealand regions for different design and detailing specifications. The results indicated that death losses for bridges were generally greater than one and downtime four times that of direct physical damage losses. It was also shown that the FAR for DAD was much less than the risk exposure to a person sleeping whereas for non-seismically designed structures, it was four and half times higher. DAD structures were demonstrated to perform really well as compared to existing ductile systems with almost same cost of design and construction. The model was demonstrated to be a useful tool in comparing structures to both scenario losses and expected annual losses.

4.2 Conclusions

The following are the major conclusions from this study:

i. The four-step risk modeling approach is shown to be a comprehensive stochastic model for estimating the 3D (death, downtime and damage) losses due to earthquakes.

ii. Death losses due to earthquakes are slightly more than damage losses. The FAR for non-seismic structure is 4.62; for seismic, it is 0.88 and for DAD it is 0.27. This shows that non-seismic structures are 4.5 times more risky than sleeping activity (FAR = 1.0). DAD structures are at least 70% less risky.

iii. The expected annual downtime loss (EADT) for all possible earthquake scenarios is at least 2 days per year for non-seismic (non-ductile) structures, whereas for their ductile seismic-resistant counterparts EADT = 0.25 days per year. Such a value for the latter may seem insignificantly small. However, in the context of a MCE (2% in 50 years) the picture is much darker: Non-seismic and seismically designed structures are expected to be down for 52 and 5 weeks respectively.

iv. Losses can be further reduced or negated if DAD concepts are employed in design and construction.

4.3 Recommendations for Further Work

This section outlines a few important areas in which further study is essential:

i. Experimental and practical investigations are required to verify the results from the model.

ii. More detail and rigorous analysis is required to confirm the better response of DAD types of structures in seismic hazards.

iii. To a greater extent, model needs more accurate and realistic relationship between damages states and 3D losses. Practical regional studies should serve this purpose where, the death loss should also depend on significance and importance of the structure (like a school building), and downtime on floor area of buildings.

iv. More detail study is required for estimating downtime cost of money as it depends on the location and it varies from owner to user point of view. For example, in case of bridges, a more detailed origin-destination network analysis is needed for reliable results.

REFERENCES

Acton, J. P. (1976). "Measuring the monetary value of lifesaving programs." *Law and Contemporary Problems*, 40(4), 46-72.

Ajrab, J. J., Pekcan, G., and Mander, J. B. (2004). "Rocking wall-frame structures with supplemental tendon systems." *Journal of Structural Engineering*, 130(6), 895-903.

Algermissen, S. T. (1972). *A study of earthquake losses in the San Francisco Bay area*, Office of Emergency Preparedness and National Oceanic and Atmospheric Administration, Washington, D.C.

Baker, J. W., and Cornell, C. A. (2008). "Uncertainty propagation in probabilistic seismic loss estimation." *Structural Safety*, 30(3), 236-252.

Basoz, N., and Mander, J. B. (1999). "Enhancement of the highway transportation lifeline module in HAZUS." *National Institute of Building Sciences*, 16(1), 31-40.

Beck, J. L., Kiremidjian, A., Wilkie, S., Mason, A., Salmo, T., Goltz, J., Olsan, R., Workman, J., Irfanoglu, A., and Porter, K. A. (1999). "Decision support tools for earthquake recovery of business." *CUREe-Kajima Phase III Project*, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.

Bradley, B. A., Dhakal, R. P., Cubrinovski, M., Mander, J. B., and MacRae, G. A. (2007). "Improved seismic hazard model with application to probabilistic seismic demand analysis." *Earthquake Engineering & Structural Dynamics*, 36(14), 2211-2225.

Comerio, M. C. (2000). *The economic benefits of disaster resistant university*, Institute of Urban and Regional Development, University of California, Berkeley, CA.

Comerio, M. C. (2006). "Estimating downtime in loss modeling." *Earthquake Spectra*, 22(2), 349-365.

Cornell, C. A. (1968). "Engineering seismic risk analysis." *Bulletin of the Seismological Society of America*, 58(5), 1583-1606.

Cornell, C. A., Jalayer, F., Hamburger, R. O., and Foutch, D. A. (2002). "Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines." *Journal of Structural Engineering*, 128(4), 526-533.

Cornell, C. A., and Krawinkler, H. (2000). *Progress and challenges in seismic performance assessment*, PEER Center News, University of California, Berkeley, CA.

Dhakal, R. P., Khare, R. K., and Mander, J. B. (2006). "Economic payback of improved detailing for concrete buildings with precast hollow-core floors." *Bulletin of the New Zealand Society for Earthquake Engineering*, 39(2), 106-119.

Dhakal, R. P., and Mander, J. B. (2006). "Financial risk assessment methodology for natural hazards." *Bulletin of the New Zealand Society for Earthquake Engineering*, 39(2), 91-105.

Department of Homeland Security (DHS). (2009). *National infrastructure protection plan*, National Institute of Building Science, Washington, D.C.

Dunster, H. J., and Vinck, W. (1979). "The assessment of risk-its value and limitations." *Nuclear Engineering International*, 15(3), 23-37.

Elms, D. G., and Mander, J. B. (1990). *Locomotive engineer hazards: A quantitative risk assessment study*, Institution of Professional Engineers, Wellington, New Zealand.

Garrick, B. J., and Christie, R. F. (2002). "Probabilistic risk assessment practices in the USA for nuclear power plants." *Safety Science*, 40(1-4), 177-201.

Gibson, S. B. (1976). "Risk criteria in hazard analysis." *Chemical Engineering Progress*, 72(2), 59-62.

Goulet, C. A., Haselton, C. B., Mitrani-Reiser, J., Beck, J. L., Deierlein, G. G., Porter, K. A., and Stewart, J. P. (2007). "Evaluation of the seismic performance of a code-conforming reinforced-concrete frame building: From seismic hazard to collapse safety and economic losses." *Earthquake Engineering & Structural Dynamics*, 36(13), 1973-1997.

Griffiths, R. F. (1978). “Reactor accidents and the environment.” *Atom*, 266(2), 314-325.

Hamid, N. H., and Mander, J. B. (2006). “Damage avoidance design of warehouses using precast hollow core walls system.” *Engineering Structures*, 9(1), 1-573.

HAZUS. (1997). *Earthquake loss estimation methodology*, Technical manual, developed by the Federal Emergency Management Agency, Washington, D.C.

HAZUS. (2003). *Multi-hazard loss estimation methodology earthquake model*, Technical manual, developed by the Federal Emergency Management Agency, Washington, D.C.

Henley, J. E., and Kumamoto, H. (1992). *Probabilistic risk assessment*, IEEE Press, New York, NY.

Jones-Lee, M. W. (1976). *The value of life: An economic analysis*, University of Chicago Press, Chicago, IL.

Kennedy, R. P. (1999). “Risk based seismic design criteria.” *Nuclear Engineering and Design*, 192(3), 117-135.

Kennedy, R. P., Cornell, C. A., Campbell, R. D., Kaplan, S., and Perla, H. F. (1980). “Probabilistic seismic safety study of an existing nuclear power plant.” *Nuclear Engineering and Design*, 59(2), 315-338.

Kennedy, R. P., and Ravindra, M. K. (1984). “Seismic fragilities for nuclear power plant risk studies.” *Nuclear Engineering and Design*, 79(1), 47-68.

Kircher, C. A. (2003). “Earthquake loss estimation methods for welded steel moment-frame buildings.” *Earthquake Spectra*, 19(2), 365-384.

Kircher, C. A., Nassar, A. A., Kustu, O., and Holmes, W. T. (1997). “Development of building damage functions for earthquake loss estimation.” *Earthquake Spectra*, 13(4), 663-682.

Kletz, T. A. (1971). “Hazard analysis: A quantitative approach to safety.” *Proc.*, *Symposium of British Institution of Chemical Engineers*, London, UK, 75-87.

Kletz, T. A. (1978). “Hazard analysis, its application to risks to the public at large.” *Reliability Engineering*, 3(4), 325-338.

Kniesner, T. J., and Leeth, J. (1991). “Compensating wage differentials for fatal injury risk in Australia, Japan and United States.” *Journal of Risk and Uncertainty*, 4(1), 75-90.

Kniesner, T. J., Viscusi, W. K., and Ziliak, J. P. (2009). “Policy relevant heterogeneity in the value of statistical life: New evidence from panel data quantile regressions.” *Journal of Risk and Uncertainty*, 40(1), 15-31.

Lawley, H. G. (1980). “Safety technology in the chemical industry: A problem in hazard analysis with solution.” *Reliability Engineering*, 1(2), 89-113.

Lawley, H. G., and Kletz, T. A. (1975). “High-pressure trip systems for vessel protection.” *Chemical Engineering*, 82(3), 81-93.

Lees, F. P. (1980). *Loss prevention in the process industries*, Loughborough University, Leeds, England.

Linnerooth, J. (1979). "The value of human life: A review of the models." *Economic Injury*, 17(1), 52-74.

Mackie, K. R., Wong, J. M., and Stojadinovic, B. (2009). "Post-earthquake bridge repair cost and repair time estimation methodology." *Earthquake Engineering & Structural Dynamics*, 39(3), 281-301.

Mander, J. B., and Basoz, N. (1999). "Seismic fragility theory for highway bridges: Optimizing post-earthquake lifeline system reliability." *Proc., Fifth US Conference on Lifeline Earthquake Engineering*, Seattle, WA, 70-85.

Mander, J. B., and Cheng, C. T. (1997). *Seismic resistance of bridge piers based on damage avoidance design*, National Center for Earthquake Engineering Research, Buffalo, NY.

Mander, J. B., Dhakal, R. P., Mashiko, N., and Solberg, K. M. (2007). "Incremental dynamic analysis applied to seismic financial risk assessment of bridges." *Engineering Structures*, 29(10), 2662-2672.

Mander, J. B., and Elms, D. G. (1994). "Quantitative risk assessment of large structural systems." *Structural Safety & Reliability*, 3(1), 1905-1912.

Mander, J., and Sircar, J. (2009). "Loss model for seismically damaged structures." *Proc.*, *2009 Structures Congress*, ASCE, Austin, TX, 1077-1086.

Miller, T. R. (2000). "Variations between countries in value if statistical life." *Journal of Transport Economics and Policy*, 34(2), 169-188.

National Institute of Building Sciences (NIBS). (1994). *Assessment of state of the art earthquake loss estimation methodologies,* FEMA-249, Washington, D.C.

Porter, K. A., Beck, J. L., and Shaikhutdinov, R. (2004). "Simplified estimation of economic seismic risk for buildings." *Earthquake Spectra*, 20(4), 1239-1263.

Porter, K. A., Kiremidjian, A. S., and LeGrue, J. S. (2001). "Assembly-based vulnerability of buildings and its use in performance evaluation." *Earthquake Spectra*, 17(2), 291-312.

Rhoads, S. E. (1978). "How much should we spend to save a life?" *Public Interest*, 51(51), 74-92.

Rice, D., and Cooper, B. S. (1967). "The economic value of life." *American Journal of Public Health*, 57(11), 1954-1966.

Shiono, K., Mubakami, H., Okada, S., and Ohta, Y. (1995). *Lessons learned from reconstruction following a disaster: Enhancement of regional seismic safety attained after the 1976 Tangshan, China Earthquake*, Japan Society of Civil Engineers, Dotoku Gakkai, Japan.

Shome, N., and Cornell, C. A. (1999). *Probabilistic seismic demand analysis of nonlinear structures*, RMS Program, Stanford University, Stanford, CA.

Sircar, J., Damnjanovic, I., Mander, J., and Aslan, Z. (2009). "Catastrophe bonds for transportation assests: A feasibility analysis." *Journal of Transportation Research Record*, 2115(1), 12-19.

Solberg, K. M. (2007). *Experimental and financial investigations into the further development of damage avoidance design*, University of Canterbury, Canterbury, New Zealand.

Solberg, K. M., Dhakal, R. P., Mander, J. B., and Bradley, B. A. (2008). "Computational and rapid expected annual loss estimation methodologies for structures." *Earthquake Engineering & Structural Dynamics*, 37(1), 81-101.

Stewart, M. G. (2009). "Risk-informed decision support for assessing the costs and benefits of counter-terrorism protective measures for infrastructure." *International Journal of Critical Infrastructure Protection*, 2010(3), 29-40.

Thaler, R., and Rosen, S. (1976). *The value of saving a life: Evidence from the labor market*, Household Production and Consumption, Columbia University Press, New York.

U.S. Department of Transportation. (2007). *Treatment of the economic value of a statistical life in department analyses*, Office of Assistant Secretary for Transportation Policy, Washington, D.C.

Vamvatsikos, D., and Cornell, C. A. (2002). "Incremental dynamics analysis." *Earthquake Engineering & Structural Dynamics*, 31(3), 491-514.

Viscusi, W. K. (1978). "Labour market valuations of life and limb: Empirical evidence and policy implications." *Public Policy*, 26(3), 359-386.

Whitman, R. V., Anagnos, T., Kircher, C. A., Lagorio, H. J., Lawson, R. S., and Schneider, P. (1997). "Development of a national earthquake loss estimation methodology." *Earthquake Spectra*, 13(4), 643-661.

VITA

Sandeep Ghorawat received his Bachelor of Technology degree from the Indian Institute of Technology Guwahati, Assam, India in May of 2008. He entered the structural engineering program at Texas A&M University in August 2008 and received his Master of Science degree in May 2011.

Mr. Ghorawat may be reached at CE/TTI, Texas A&M University, 3136 TAMU, College Station, TX 77843-3136, USA. His email is ghorawatiit04@gmail.com.